September 21, 1933

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Engineering News-Record

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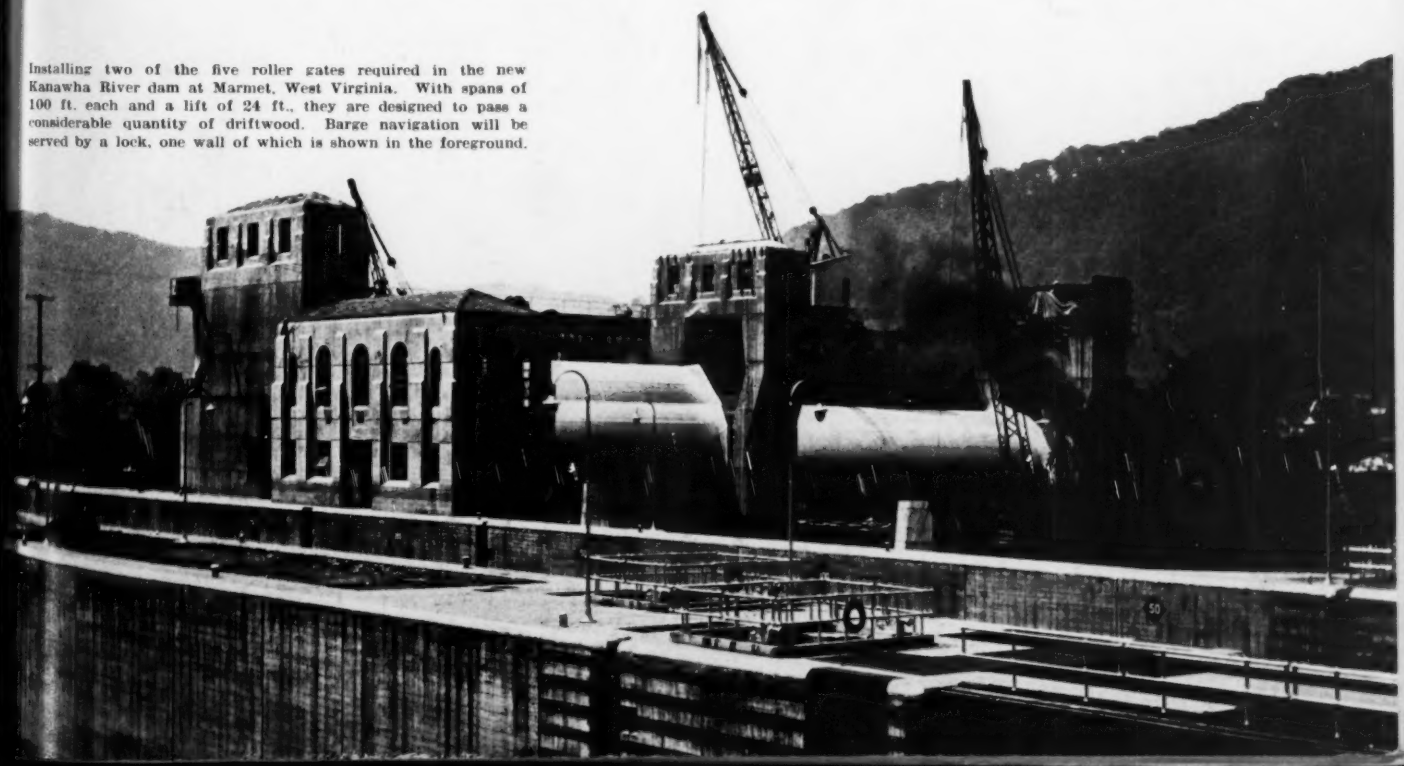
Roller Gate Dams for River Improvements

A new development in American dam practice for slackwater navigation is represented in the roller-gate structures now being installed at several locations and planned for use on the Upper Mississippi work for which the PWA appropriated $22,000,000 last week. Particularly adaptable to streams subject to protracted periods of low water and severe ice conditions, the new dams promise to be standard equipment for many river navigation improvements. The dam on the Kanawha River at Marmet, W. Va., described in this issue, is typical of the type.

ALSO IN THIS ISSUE:

A Survey of Employment and Values in Construction Industry



Installing two of the five roller gates required in the new Kanawha River dam at Marmet, West Virginia. With spans of 100 ft. each and a lift of 24 ft., they are designed to pass a considerable quantity of driftwood. Barge navigation will be served by a lock, one wall of which is shown in the foreground.

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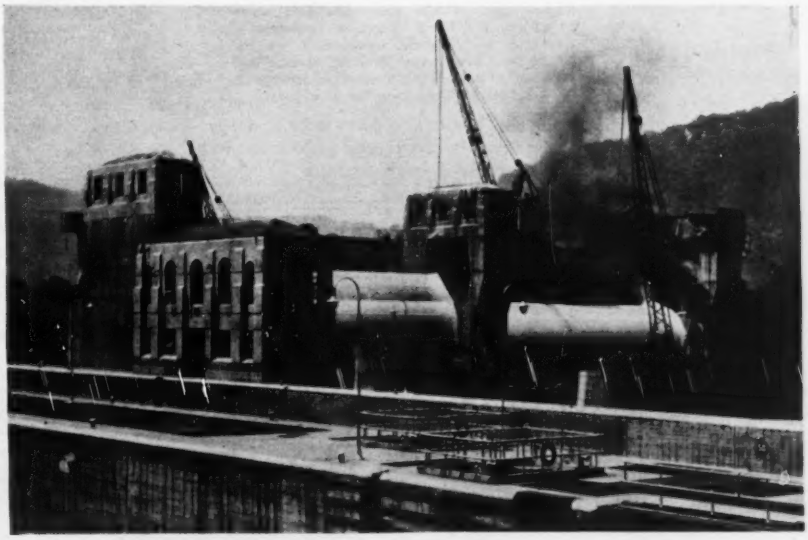
Roller-Gate Dams for Kanawha River

Two replacement lock-and-dam structures represent recent recommendation by army engineers of non-navigable type of movable dams for important slackwater navigation improvements

TWO lock and dam operations on the Kanawha River in West Virginia are inaugurating both a navigation works replacement program on this important little tributary of the Ohio and a new development in navigation-dam practice in the United States. The new development is the adoption of the non-navigable roller-gate type dam extensively used in Europe, particularly in Germany and in the Scandinavian countries, but one that is represented in the United States by only a few small structures, notably by the dam at Bellows Falls, Vt. (*ENR*, Dec. 8, 1927, p. 908), and the Grand River Dam in Colorado (*Engineering News*, July 6, 1916).

Following the report in 1931 of the board of engineers of rivers and harbors, recommending the new type of dam for the 9-ft. navigation improvement of the upper Mississippi, a dam was put under construction on the Mississippi at Rock Island, Ill., and two more on the Kanawha River in West Virginia in the project to revamp the 35- to 50-year-old navigation locks and dams of this stream on which business is increasing. A dam of the roller-gate type is now planned for the new navigation works at Montgomery Island on the Ohio River. This also is a project to replace several old navigation structures by a single larger dam. With the upper Mississippi project approved, as now seems probable, among projects to be undertaken under the National Industrial Recovery Act, other dams will rapidly arrive at the active stages of design and construction.

Fig. 1—View taken from the land wall of lock, looking upstream and riverward and showing progress of work on the Marmet Dam. Roller gates 1 and 2 are being given final tests.



Until the above-mentioned dams were undertaken, army engineering practice had adhered nearly universally to navigable types of dams, permitting open-river navigation at the higher river stages. Practice on the Ohio River, with its half a hundred navigation dams and locks, may be taken as representative. Here the typical navigation works comprise a lock or locks at one bank, a navigable section of at least 600 ft. of Chanoine wicket dam, a non-navigable weir section of variable length and bear-trap gates and wickets and a fixed section, generally short—all sections carrying the barrage from the locks to the opposite bank. Chanoine wickets, first used with bear-trap dams in the non-navigable section, were failures, and Rebout wickets were afterwards used. These in turn have not given full satisfaction. Now the bear trap is the main dependence for pool regulation. With wickets lowered, the navigable section gives open river navigation whenever the river stage provides a depth in

excess of 9 ft. over the sill. Both it and the collapsible weir section give exceptional floodway. On the Ohio River these qualities are important to meet the great range of stages and the considerable duration of stages high enough to permit open river navigation.

As in the case of the Ohio, wherever in America in the past slackwater navigation has been undertaken, consideration of floodway and more particularly provision for open river navigation have been held dominant. Thus the navigable wicket-type dam is nearly universal on rivers long improved for this kind of navigation. Of the ten old dams, on the Kanawha two at the head of navigation are fixed and the eight others are Chanoine wicket structures. (The two roller gate dams described in this article replace two of the wicket dams and the two fixed structures.) In general, the navigable movable dam is traditional American practice. The change of type for the upper Mississippi and other locations requires explanation.

The turn to non-navigable dams is less a reversal of practice than it is a logical adaptation of type to river regimen and navigation conditions. The situation is best comprehended by considering the argument of the special board of engineers as approved by the board of engineers of rivers and harbors in the case of the upper Mississippi. In this argument contrast is made with the Ohio River, with which the upper Mississippi project and its proposed 28 lock and dam works is most nearly comparable in magnitude.

As previously indicated, the extreme range of high- and low-water stages (as great as 69 ft.) on the Ohio River and the long duration of stages permitting open river navigation influenced the adoption of the wicket-type dam that would lower onto its sill and provide unobstructed discharge for floodwaters and free passage for navigation. The hydraulic characteristics of the upper Mississippi do not impose such requirements; flood heights are not great and stages sufficient for open river navigation are short. A type of movable dam operated from a fixed bridge is entirely practicable. Its operation is cheaper, more dependable and less hazardous than that of wicket dams, which must be operated from a maneuver boat and require a greater number of skilled operators, often working under hazardous conditions. Repairs are more easily made. This type of dam permits gate types that reduce hydraulic jump and downstream scour and facilitate the passage of ice and drift.

In the upper Mississippi plans two types of movable gates (tainter and roller) were recommended to be used in combination, the dam to consist "mainly of tainter gates with a sufficient number of roller gates to pass running ice and drift and to provide ample capacity for the greatest flow likely to occur during the winter months." A length of 100 ft. was recommended as standard for the roller gates. (This has since been changed to 60 ft. for pile foundations.)

Enlarging on gate operation under ice conditions, the report said: "In order to maintain pools throughout the year in the section above Rock Island, manipulation of the gates during freezing weather is essential. The tainter gates require expensive heating arrangements to make it possible to operate them during the winter, and even then dependability of operation is not positively assured. The roller-gate type of dam offers the greatest assurance of operation under ice conditions, with the maximum of clear opening between piers. This type has not been used extensively in this country. It has been in successful use abroad for many years, especially in the Scandinavian countries, where operation under ice conditions far more severe than is expected on the upper Mississippi is a regular thing."

The use of roller gates is apparently controlled by basic patents throughout the world, held by the Krupp company and the M. A. N. company of Germany. The American rights are held respectively by the Dravo Contracting Co., Pittsburgh, and the S. Morgan Smith Co., of York, Pa. Of the dams named previously as now under construction, the two at Marmet and at London on the Kanawha River are of the Krupp type, and the one at Rock Island is of the M.A.N. type.

Epitomizing the reasons for selecting roller-gate designs for the dam at Rock Island and those on the Kanawha River, a statement from the office of the engineer of the Upper Mississippi Valley Division, Corps of Engineers, U.S.A., contains the following:

> Dam No. 15, Mississippi River, at Rock Island, Ill., was the first structure designed by this department with roller gates. It was essential at this site that the type of gate selected be rugged and positive in operation during severe cold weather, and of large span to assure passage of great masses of floating ice during the spring break-up. The roller type of gate, which has been developed and extensively used in Europe, has demonstrated its successful use under such conditions. The inherent strength of the gate drum permits of relatively great span with economical use of metal. The massive construction of both gate and operating machinery assures positive operation under severe physical and climatic conditions which cannot be positively foreseen and for which no accurate mathematical analysis can be made in the design.
>
> The design of the gate lends itself well to relatively inexpensive provisions for heating both the gate drum itself and the side seals for operation during continued freezing weather. At Dam No. 15, compartments at both ends of the lock drum are provided with simple electric strip heaters of standard manufacture to free the roller track and main pier recess from ice. The steel-plate liners on the faces of the piers also are provided with small recesses in which strip heaters are placed to free the side seals from ice.
>
> In selecting the type of dam for the Kanawha River, consideration was given to the characteristics of the river. This river flows through a narrow valley bor-

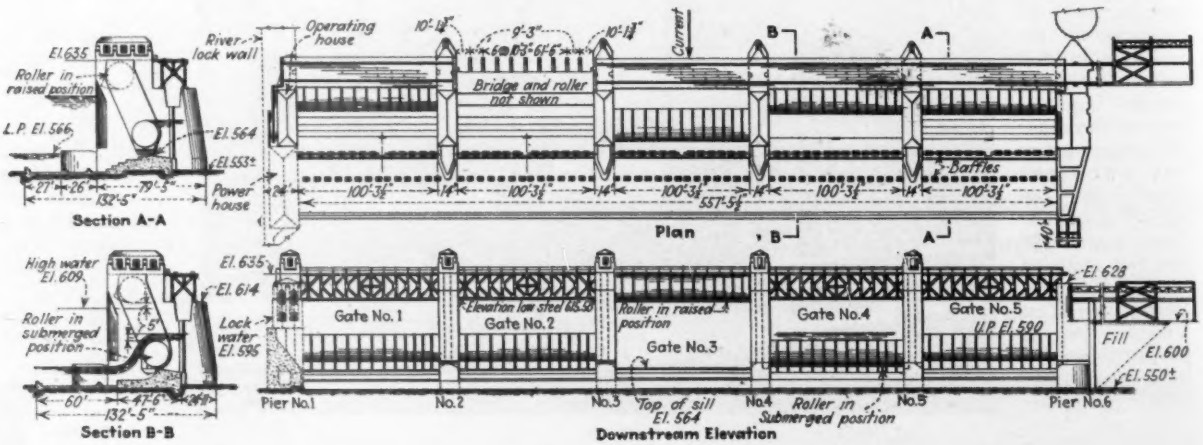


Fig. 2 — Plan and elevation of Marmet Dam, one of the first roller-gate dams built by the U. S. Engineer Department. Marmet and London dams, both on the Kanawha River, are identical in size and design. One of the five roller sections is submergible to pass ice and logs.

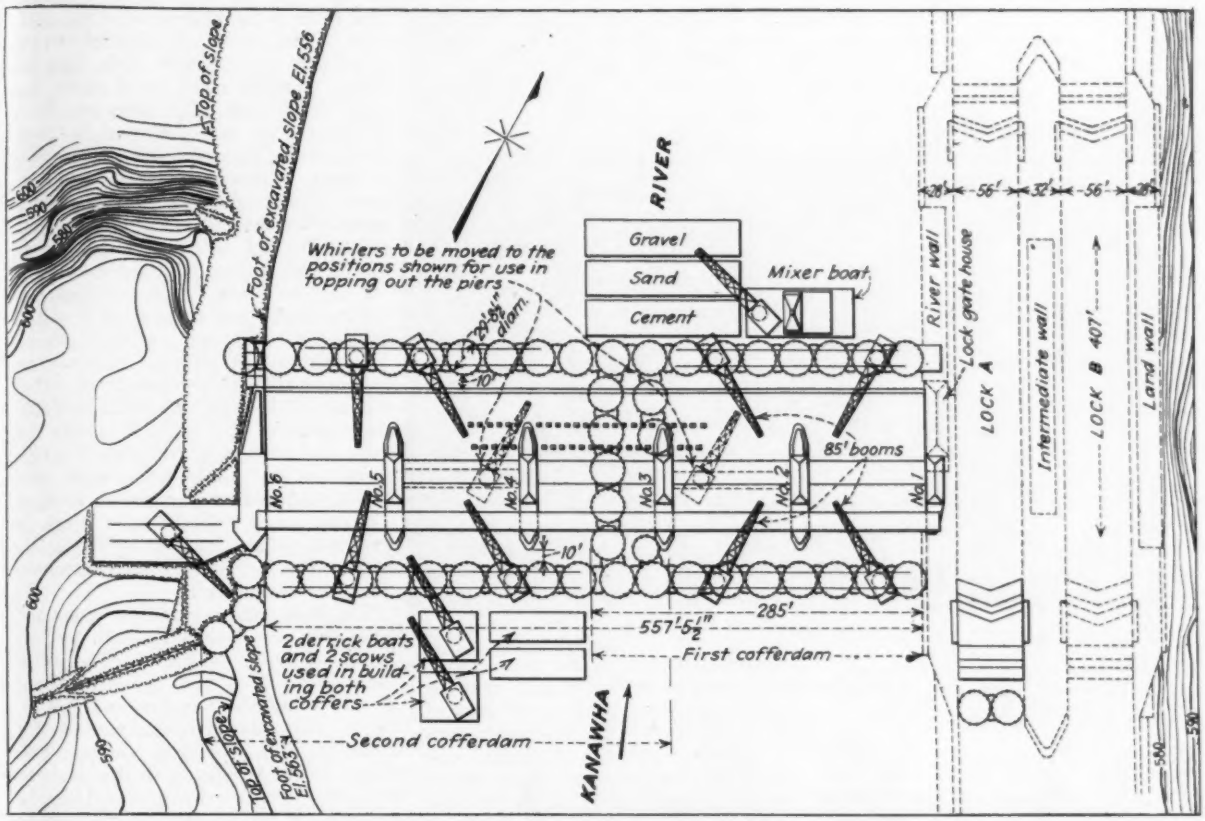


Fig. 3—Construction layout and cofferdam design at Marmet Dam. Heavy plant consisted of four whirler cranes on the cofferdam, two on derrick boats and a complete mixer boat. The east half of the dam will be completed in the first cofferdam section before the second half is started within the second cofferdam section.

dered by high steep hills. The flood plain, which extends to the foot of the hills, averages about one-half mile in width, and the river bed varies from 700 to 900 ft. between banks. These banks are pronounced and rather steep. The bed of the river is approximately 50 ft. below the top of the banks. The low-water slope averages about 2 ft. per mile.

Since the drainage basin lies entirely in a mountainous region, the river is subject to rapid run-offs with rises of 15 ft. or more in 24 hours. Maximum stages reach from 45 to 50 ft. above low water but are of brief duration.

The narrow valley and lift of 24 ft. selected for the dams at Marmet and London on the Kanawha necessitate a dam of the movable type with low sill and wide pier spacing in order to provide sufficient discharge area during flood stages so as not materially to increase high-water level.

There are a number of types of movable dams, such as tainter gate, vertical lift, roller gate and sector gate, which are applicable to such conditions. While little difficulty from ice is expected in operating the Marmet and London dams on the Kanawha, a large amount of heavy drift is carried by this stream during floods, including occasional large trees. A span of 100 ft. is considered advisable to pass such a drift. The wide spacing also is desirable in order to reduce to a minimum the area occupied by the piers. For this long span and the lift of 24 ft., the roller type of gate, with its record of dependable operation in European countries and in the few installations in this country, was considered the most dependable and satisfactory. In future designs of varying dimensions the vertical lift or other types of movable gateway may be found advantageous.

In the following text and illustrations reference is confined to the Kanawha dams, as they are farthest advanced in construction.

Kanawha dams

The new dams on the Kanawha are replacement structures; they take the place of four locks and dams constructed between 1880 and 1887. At present, the river is improved for a little over 90 miles above its mouth at Point Pleasant, W. Va., where it enters the Ohio River. The existing works are ten locks and dams, the latest of which was completed in 1898. As stated, the new works, one at London (mile 83) and one at Marmet (mile 68), will replace four of the old dams, two of which were fixed and two were movable, of the Chanoine wicket type, built 53 years ago. The two Chanoine wicket dams at Marmet (No. 5) and Cabin Creek (No. 4) were the first to be built for navigation purposes in this country. Their present condition shows that 50 years is long enough for a movable dam to be operated.

The new locks and dams at London and Marmet are identical structures, and as the same contractor is building both, the methods of construction are alike. Only the structures and operations at Marmet will therefore be described. These are indicated in plan by Fig. 3. Briefly, the structures are twin locks on one back, flanked by an across-stream dam consisting of five roller gates between concrete piers. A plan and elevation of the dam structure are shown by Fig. 2.

The structure and construction of the dam are principally of interest. In general, the lock design and construction followed standard practice. At the beginning of the present contract the river-lock walls had been completed except for the gates and mechanical equipment. The General Contracting Co., Pittsburgh, Pa., built lock A at Marmet, and John Walsh, Northport, N. Y., (Long Island) built lock A at London; Locks B at both places were built by the present contractor for the dam. The present lock work included the land wall of the land-side lock and the gates and mechanical equipment for both locks. These were installed for the river lock as the first operation, which provided for navigation while the dam construction and land-side lock work was proceeding. The view, Fig. 4, shows the river-side lock walls as taken over and the land wall of the land-side lock under construction. The view shows operations as flooded out during

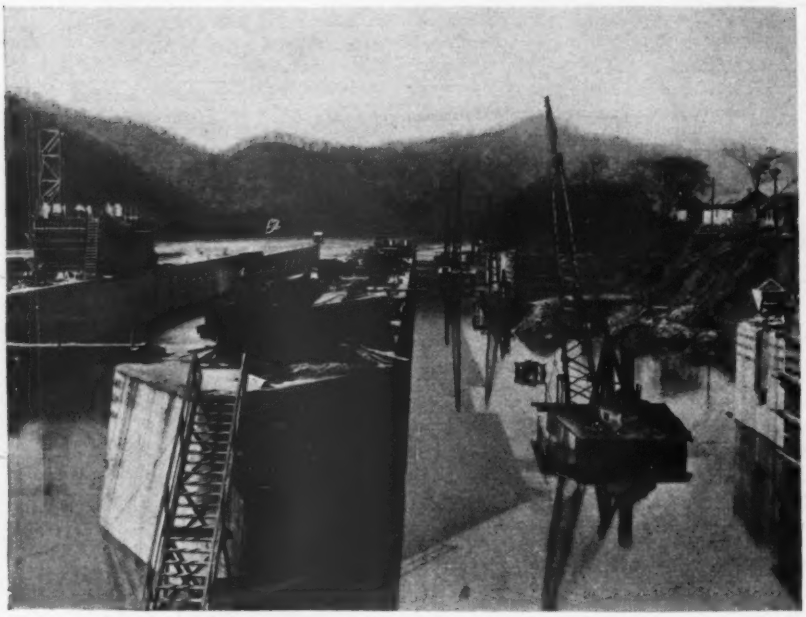


Fig. 4—Floods interrupt work of building the land wall of twin locks at Marmet Dam on the Kanawha River in West Virginia. Four large steam cranes were submerged when the cofferdam was flooded.

a rise of the river in March, 1932. The locks are of the standard Monongahela River size (56x360 ft. in the clear).

The dam structure and dimensions are shown by Fig. 2. Aside from the gates, which are described farther on, the essential parts are the piers recessed to provide for the bulkheads and the roller gates, the sill on which the gates take seat, the apron with baffle piers to break up the flow and the operating bridge. The first two gates (Fig. 1) are now completed and were placed in operation on Aug. 7, 1933, the contractor's cofferdam serving as the remainder of the dam. The old wicket dam is laid down and no longer used. Incidentally it is interesting to note that the gates of the old lock at Marmet (No. 5) are of wrought iron, have served 50 years and are still in serviceable condition. As an experiment the flap-type roller in the new dam is also to have wrought-iron plating.

Operating structure

Adoption of roller-gate dams on the Kanawha, and for other recent installations as noted, is based in general on the condition that the provision of open-river navigation is not of sufficient importance to outweigh the advantages of this type, and on the facts that the non-navigable dam is more dependable, less hazardous to operate and can be repaired more readily and with less hindrance to navigation. The long gate clearance for passing ice floes and drift is also an advantage where these occurrences have to be considered. On the Kanawha the influencing conditions were the comparative unimportance of open river navigation and the operating advantages of the roller-gate dam, but special provision for ice was made at one of the gates. One important reason for adopting roller gates rather than sector gates was the fact that the sill had to be within 6 ft. of the riverbed to obtain sufficient discharge capacity in floods. This prevented the use of sector gates, which could not be lowered without making an excavation in the riverbed, which would fill up with sediment.

As shown by Fig. 2, the new dam is 557½ ft. long from the river wall of the new locks to the abutments on the other side of the river. It consists of five roller-gate spans, 100 ft. 3½ in. long between concrete piers 14 ft. wide and about 85 ft. high. The piers are 79 ft. 5 in. long at the base, with a 26-ft. downstream extension above low-pool elevation. Between the piers are concrete sills for the gate and a concrete apron with baffle piers extending downstream for the sills for a distance of 60 ft., with stoplog facilities for closing off the lower pool. Across the tops of the upstream ends of the piers is a steel bridge for carrying traveling cranes. The bridge will carry a traveling revolving crane on the upper level with a capacity of 30 tons, which will facilitate the passage of drift and will service the gates and machinery, and a small bridge crane on the lower level with twin hoists, each with a capacity of 15 tons. The emergency bulkhead units weigh 28 tons and will be handled by the lower crane. Fig. 2 shows the various elevations of the dam parts.

The design and operation of gates and bulkheads are best visualized by the several drawings of Fig. 6. The roller gates are hollow steel cylinders about 20 ft. in outside diameter and 108 ft. 5½ in. long, braced inside and having shells of $\frac{3}{16}$-in. and $\frac{5}{8}$-in. plates. The drums do not fit directly on the concrete seal when in a closed position. There is an apron sector attached to the outside of the gate, which acts as a seal between the concrete sill and the drum proper. This apron is curved and attached to and rolls up with the drum.

On each end of each drum is a sprocket; it engages an inclined rack set in the side of the piers for the raising operation. The rollers are raised by machinery set in the top of each pier, which operates a double-link chain wrapped around the drum. As the gates are hoisted, the surplus chain is auto-

Fig. 5—One corner of the first cofferdam at Marmet Dam, built up of full circular-cell units of sheetpiling, filled with sand and gravel and paved with concrete. Four large whirler cranes operate on wide-gage tracks laid on the paving.

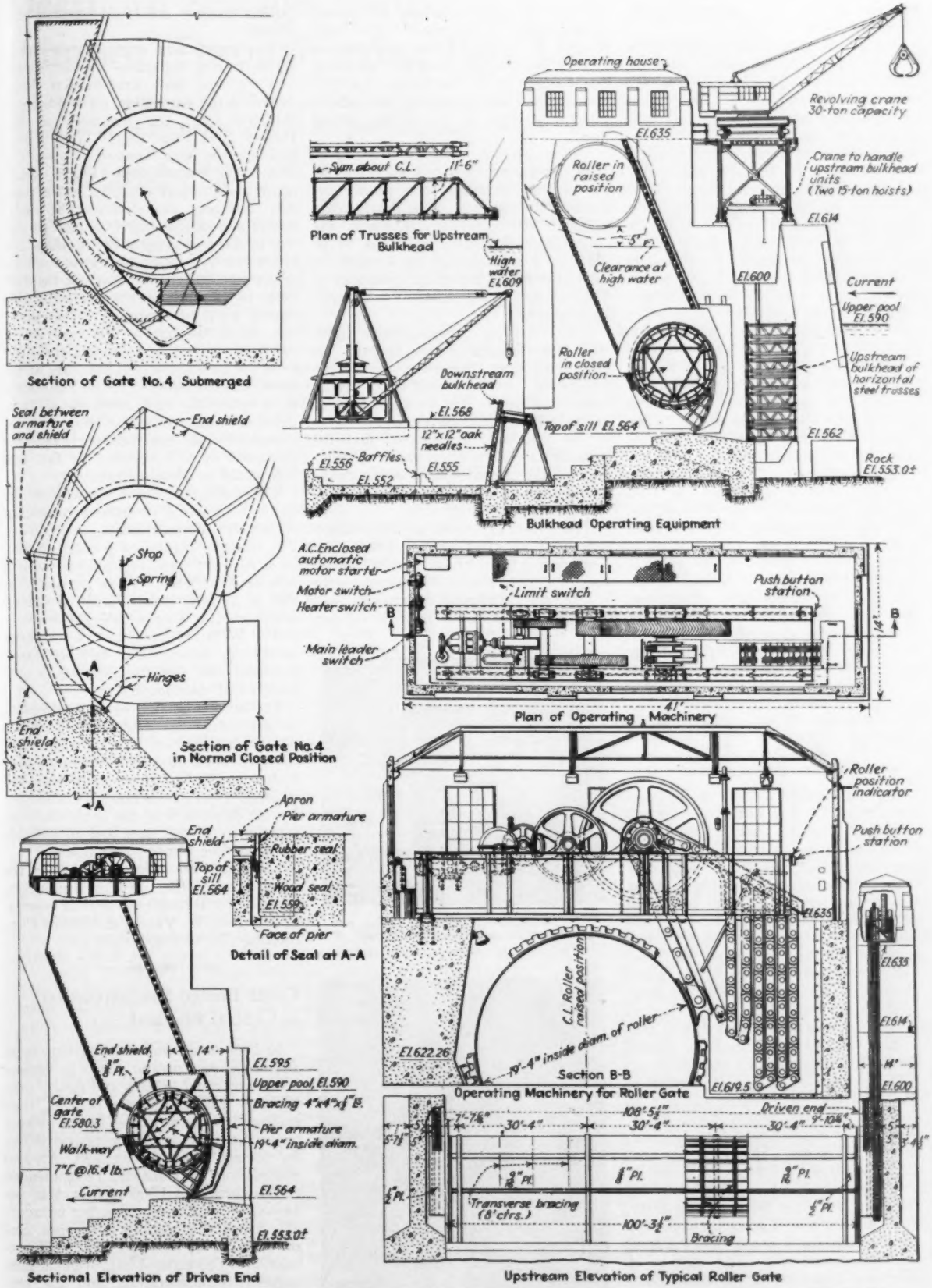


FIG. 6—STRUCTURAL AND MECHANICAL DETAILS OF ROLLER GATES AND BULKHEADS OF KANAWHA RIVER DAMS

Note: Submergible gate No. 4 as shown in upper left hand corner was not used as the contractor submitted an alternate design, acceptable to the engineers, providing for a smaller diameter drum with a hinged flap on the top side which can be folded down over the drum to create a floodway 5 ft. in depth.

matically racked in a recess at the top of the pier.

The drum of one of the five roller gates is of smaller diameter than the others and is provided at the top with a movable flap hinged to it. This flap is actuated independently of the gate proper and can be lowered onto the drum to provide a maximum depth of overflow of 5 ft. for the purpose of passing surface drift and debris. The gates are raised from one end only. The other end of the drum rolls up the rack, but no power is applied at that end. The operating mechanism for each gate is independently controlled. A 60-hp. electric motor does the lifting through worm and other reduction gears. The rollers raise a maximum vertical distance of 43.4 ft. at a speed of 9 in. in a minute. A rubber and wood seal is provided between the pier and the end of the roller. A wood seal is also provided between the concrete sill and the apron of the roller, which seats on a steel H-beam embedded in the concrete.

The upstream bulkhead units are made up as horizontal trusses 4 ft. 1 in. high, timber-faced on the downstream side; the trusses are 11½ ft. back to back of chords. The trusses are made up of structural nickel steel with an ultimate strength of 96,000 lb. per sq.in. Nickel-steel, ⅞-in. rivets are used throughout. Each unit weighs 28 tons. Seven units are required for each of the five openings. The downstream bulkhead for closing off the lower pool consists simply of vertical 12x12-in. oak needles set against the structural-steel frame. These needles will be handled by a derrick boat.

Construction methods

The outstanding feature of the construction of the dam is the circular-cell cofferdam (Fig. 3). Construction operations are in two parts: first, the construction of half the dam toward the lock; and second, the completion of the dam across the river. The cofferdam for the first half is 285 ft. long and about 200 ft. wide. It includes the two spans of the dam adjoining the river wall of the lock and two of the piers. The second section is about 272 ft. long, with the end wall of the first cofferdam section torn out and replaced with cofferdam sections tying into the second pier built within the first cofferdam.

The cofferdam design is similar to that used by the same contractor at Deadman's Island Dam on Ohio River, described in *Engineering News-Record*, Jan. 9, 1930, p. 66. The cofferdam consists of sheetpile cylinders 29 ft. 8½ in. in diameter built up of 27-ft. sheeting. The river bottom of the Kanawha at the Marmet Dam site is ideal for this type of cofferdam, for it is practically a level rock formation. The first section of cofferdam contained 23 of these complete cylinders each connected to the adjoining sections by two arc sections. The cells and spaces within the arc connections are filled with sand and gravel dredged out of the river. The only change of design of the cofferdam as built at Marmet from that built at Deadman's Island is the installation of a 6-in. concrete paving on top of the cells to prevent erosion of the filling during floods.

Steam whirler derricks with 85-ft. booms operating on wide-gage tracks on top of the cofferdam, two on the upstream side and two on the downstream side, are used to excavate within the cofferdam. Wherever rock has to be removed it is broken up fine enough for the clamshell bucket of the excavator to handle. Excavated material is dumped into barges.

The cofferdam is remarkably tight, with more leakage coming through the rock itself than through or under the cofferdam. A 10-in. electrically driven centrifugal pump has been found adequate to keep the chamber dry.

The specifications required that the trench of the foundation under the concrete sill be cut out square with a channeling machine. This machine in operation is shown by Fig. 7.

The Francois system of cementation was used in grouting the foundation. The process is to drill a hole 10 ft. deep and force grout through it, starting with a thin and gradually increasing to a thick mix. The hole is then drilled out, drilled deeper, and the grouting repeated.

Fig. 7—Channeling machine cutting out key trench in rock foundation for sill of dam.

Fig. 8—Periscope developed by U. S. engineers permits examination of foundation formation by means of drill holes.

The government engineers developed an interesting apparatus for the inspection of test and examination holes drilled in the foundation. The idea was obtained from the inspector at the Hawks Nest Dam on the New River, built for the New-Kanawha Power Co. The device is a periscope (Fig. 8) made up of a 2-ft. piece of 2½-in. stove pipe, two mirrors, an electric-light bulb, socket and electric wire. A part of one side of the pipe is cut away to allow one of the mirrors, set at 45 deg. to the axis of the pipe, to reflect the side of the drill hole; the light is suspended below this mirror with the other mirror set at 45 deg. below the light. The function of the lower mirror is to reflect the light from the bulb above it to the sides of the holes, and the function of the top mirror is to reflect the light from the sides of the hole to the eyes of the observer. By means of this ingenious periscope the formation of rock revealed by the drill hole could be closely examined.

The cofferdams were constructed by means of floating equipment consisting of whirler derrick boats, scows, etc. The concrete plant is a permanent unit on a steel hull and is equipped with a whirler derrick for feeding the steel bin and a 2-cu.yd. mixer with weighing equipment for proportioning the materials. With the exception of weighing equipment, boilers, and drilling equipment, all plant is of the contractor's own design and manufacture.

The second section of the cofferdam consists of 24 complete cells. In addition thereto, there will be three permanent cells connecting the abutment pier with a single line of sheetpile wall 150 ft. long; this construction was necessary for the protection of the bank after the completion of the dam and to provide for a possible future power house.

The dams are being built under the supervision of the Corps of Engineers, U. S. Army, through the district office at Huntington, W. Va., by the Dravo Contracting Co., of Pittsburgh, Pa.

Canal Bridge Reconstruction in Central England

In 1929 an act was passed giving local authorities in England power to assume control of canal bridges and finally prohibiting traffic over the bridges, thus ending the impasse caused by the owners of canals in disclaiming responsibility for damage caused by accident. A royal commission on transport recommended reconstruction of 1,000 such bridges per annum by local authorities, but unfavorable financial conditions intervened, and the Minister of Transport withheld permission. The general improvement now permits reconsideration, and the embargo has been lifted. The Birmingham Chamber of Commerce is urging local authorities to take action.

Dearborn Sewage Sludge Pumped Through Long Pipe Line

Line of 8-in. cast-iron pipe shows a Hazen-Williams coefficient of 126 when conveying sludge of 0.5 per cent solids content

By George E. Hubbell
Hubbell, Hartgering & Roth, Inc., Detroit, Mich.

ONE important link in the sewage-disposal system for the city of Dearborn, Mich., is the cast-iron sludge line, 20,466 ft. long, connecting the existing Imhoff tank plant at Miller Road with the new West Side plant recently completed. (See *ENR*, Feb. 18, 1932, p. 252.) Under an agreement with the city of Detroit, Dearborn is giving primary treatment to Detroit's Southfield Road district sewage at the Miller Road plant. This was accomplished by converting the Imhoff tanks into plain-sedimentation tanks which are operated with sludge removed daily. The sludge is pumped to the new West Side plant for dewatering, thus concentrating all the sludge-dewatering operations at the new plant.

Sludge Line—The line is built of 8-in. class 240 cast-iron pipe laid to grade with a minimum introduction of bends. Inspection manholes are placed about every 550 ft. along the line to facilitate maintenance. A gate valve, a gage-test cock and a removable pipe section are located in each inspection manhole. Air valves are installed at all high points and have operated satisfactorily. At each low point a blow-off connection to an adjacent sewer permits flushing of the line.

The sludge pumped is obtained from the sewage of a combined system. The sewage passes through coarse bar screens, grit chambers and the converted Imhoff tanks. The design called for a minimum pipe-line velocity of 2.5 ft. per second, with a maximum velocity of 4.0 ft. per second. At present, the operating velocity is approximately 2.8 ft. per second.

Sludge Pumps—The pump installation consists of two identical non-clogging centrifugal sludge pumps, each capable of delivering 350 g.p.m. against a head, including friction, of 120 ft.; and when operated in series, of delivering 500 g.p.m. against a head, including friction, of 220 ft. The characteristic curve of the pump is also such that one pump operating alone delivers about 500 g.p.m. against a total head of 110 ft.

These requirements were made to give flexibility in operation and discharge to meet the varying conditions of the pipe line and sludge. The pumps are driven by 30-hp. squirrel-cage motors operating at 1,800 r.p.m. The over-all efficiency of the pumps and motors is 50 per cent.

Individual 6-in. sludge suction pipes run from the Imhoff tank pots to the sludge manifold from which the pump suction is taken, the longest suction line being 350 ft. The pump suction is under a positive static head of 13.5 ft. Means are provided for backflushing the suction lines with settled sewage pumped by the sludge pumps. Settled sewage is also used for the flushing of the main sludge line.

To date, there has been one minor stoppage of the main 8-in. sludge line. This stoppage was easily removed by series-pumping at a pressure of 140 lb. as compared to normal operation of the pumps singly at 51 lb. Present operation consists of pumping sludge with an average water content of 99.1 per cent for a period of sixteen hours per day. Pump-discharge pressure readings, taken hourly, vary from 45 to 55 lb. This is due to the variation in the character of the sludge.

Test of pipe line

On the completion of the line, tests were conducted to determine the clear-water friction coefficient of the main. An unusually fine set of conditions existed for making this test, as the line is continuous and contains only four 90-deg. bends and six 45-deg. bends. Pressure readings were taken at intervals along the line with a calibrated test gage, readings being taken to the nearest tenth of a pound. Volumetric measurements were made of the discharge, and all results were reduced to the friction coefficient in the Hazen and Williams formula.

With one pump in operation, the discharge was 504 g.p.m. against a head of 111.2 ft., giving a velocity of 3.15 ft. per second. The measured friction loss was 4.33 ft. per thousand feet, which gives a coefficient of 140.

With two pumps in series, the discharge was 636 g.p.m., giving a velocity of 4.06 ft. per second. The measured friction loss was 7.1 ft. per thousand feet, which gives a coefficient of 137.

These results are in close accord with the valuation given by Hazen and Williams for the very best cast-iron pipe, and indicate the class of construction work performed.

Shortly after the above tests, a complete test of the line was made while pumping a sludge having an average moisture content of 99.5 per cent. The discharge was 465 g.p.m., giving a velocity of 2.96 ft. per second. The measured friction loss was 4.54 ft. per thousand feet, which gives a coefficient of 126.

No direct conclusion as to the variation of C with the variation in solid content of the sludge can be drawn from the records of operation because of the constantly changing character of the sludge pumped, which gives rise to a highly non-uniform solid-content distribution throughout the length of the pipe line. However, the average pump discharge has been 415 g.p.m. against an average head of 50.9 lb. per sq.in., as shown by the operation records (hourly pressure readings of the pump discharge). These data give an average operating value for C of 110 for the pipe line, the average solid content of the sludge pumped being 0.9 per cent, as shown by daily analysis of a composite sample made up of hourly samples.

Water Power Available in Santiam River Basin, Oregon

The total potential water power of the Santiam River Basin in Oregon, without storage, is 143,000 hp. for 90 per cent of the time and 447,000 hp. for 50 per cent of the time, according to figures of the United States Geological Survey. With storage at proposed reservoir sites, these figures would be increased to 351,000 and 492,000 hp. respectively.

The waters of the North Santiam River have been used for the development of power since 1866, when a power canal was built at Stayton. No large plants have been built in the basin, but there are eighteen small plants with a total installed capacity of 4,830 hp. Five of the plants are on one canal at Stayton, and seven more use water diverted to Mill Creek from the North Santiam River.

These and other pertinent facts are set forth in a typoscript report which includes 205 pages of text and illustrations as well as numerous maps. It describes in detail the water-power resources of the Santiam River Basin and the geology of the principal dam sites.

The report proposes fourteen power sites on the North Santiam River, two on Marion Fork, Puzzle Creek and Pamelia Creek, eight in the basins of the Breitenbush River and Little North Santiam River, and twelve on the South and Middle Santiam Rivers; also seven reservoirs with a total storage capacity of 828,000 acre-ft. Reconnaissance investigations of the geology at many of the dam sites were made, and the approximate depth of overburden was determined at one site by geophysical methods.

The report may be consulted at the district office of the Geological Survey in Portland, Ore., or at the main office in Washington, D. C.

Girderless Flat-Slab Bridges for Grade Crossings

A review of experience on the D.L.&W. Railroad indicates that substantial economies are possible with this type of structure — Savings result from relatively thin slabs, simple formwork and reduced excavation or fill

By M. Hirschthal
Concrete Engineer, Delaware, Lackawanna & Western Railroad Co., Hoboken, N. J.

THE girderless flat slab of reinforced concrete has proved to be of great value to railroad-bridge designers, particularly in solving many of their difficult grade-crossing elimination problems. The reasons for this, aside from the economy of the structures themselves, are first that, because of its shallow floor-slab requirement, it is possible to provide an undercrossing in locations where an overcrossing would otherwise have to be resorted to; second, by the same token, grades of freight lines can be held within the required maximum limits without costly changes such as lengthening of track and excessive grading, particularly in the vicinity of freight yards; also in the case of a series of grade eliminations, as in cities, there is an additional reduction in the grading work and in the required retaining walls between individual bridges as well as in property damages at the various streets. Aside from these advantages, the girderless flat slab is adaptable to all manner of skews and grades, it is easy to construct, and its appearance is pleasing. Although girderless flat slabs have been used for some time in building construction, their adoption for general use in railroad track structures is of rather recent date. The Delaware, Lackawanna & Western Railroad has had extensive experience with girderless flat slabs, and this article is planned to draw on that experience extensively.

Historical development

In 1913 the attention of the engineering department of the Lackawanna Railroad was attracted to the construction of the Soo Terminal in Chicago, a large freight structure of the girderless flat-slab type and probably the first railroad structure to utilize such a design. The two principal improvements then under consideration by the Lackawanna were a passenger terminal at its western terminus at Buffalo, N. Y., requiring a station viaduct, and a grade-crossing elimination project and new station at South Orange, N. J. An investigation of the new type of design in comparison with other designs in both reinforced concrete and steel disclosed that girderless flat slabs had distinct advantages in first cost, simplicity of formwork, uniformity of construction and reduced thickness of floor depth, resulting in less approach work. In general, it provided a more stable and flexible type of structure to resist railroad impacts. These advantages led to the adoption of the girderless flat-slab design for both the projected viaducts, the one at Buffalo for the slow movements at a terminal, the other subject to non-stop express traffic.

Design methods

The girderless flat slab is a development of the two-way slab supported on four sides by girders, an arrangement equivalent to a flat plate supported at its four edges. To eliminate the formwork entailed in the offsets of the girders below the bottom of slabs, the girderless slab was devised, with and without drop-panels. In effect, it is a flat plate supported at its four corners.

Inasmuch as this system was first used for buildings, the coefficients obtained (which were from time to time modified by tests) apply to conditions in which loads can be arranged in alternate panels to satisfy requirements of maximum moments. Bridges subject to moving loads cannot be treated in this manner, and a different design procedure must be followed. While the writer does not desire to go into the detailed analysis of the girderless flat slab, he wishes to point out the methods of approach available for that purpose. In effect, this design with column flares, drop-panels and heavy longitudinal reinforcement bent into the slab, is similar to that of any rigid frame in which the vertical members (columns or piers) relieve the horizontal members (beams, girders or slabs) of their moments, the amonut of relief depending upon the relative moments of inertia.

There are three methods of analysis of this type of slab: (1) the cantilever method; (2) the continuous-beam method; and (3) the fixed-beam method.

Cantilever Method—This method of analysis is based on the assumption that the reinforced-concrete plate consists of two parts, one a circular or nearly circular plate extending from the center of a column to the line (or curve) of contraflexure or zero moment, along which line the second part of the slab, by means of its shears or reactions, is hung, so that the negative moment at the edge of the column capital is that resulting from the superimposed load over the cantilever plate and that concentrated along its edge. The positive moment in the remainder of the various sections is that due to the simple-beam action of a span equal to the distance between the lines of contraflexure.

Fig. 1—Girderless flat-slab grade separation of D.L.&W. Railroad and Broad St. in Clifton, N. J. With 24-in. slab thickness and 8-in. drop panels, this structure carries four main-line railroad tracks and is typical of what can be accomplished with this type of design.

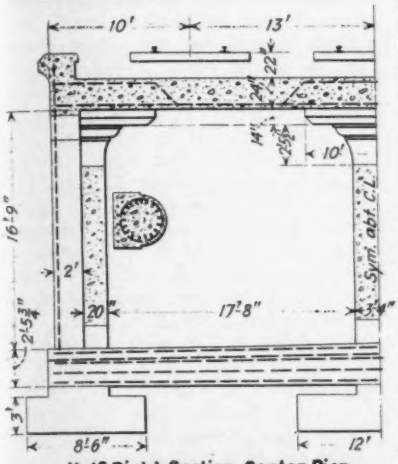
While the formulas derived from this condition are complicated, the fact that the radii of the column capital and of the lines of contraflexure are given as a fractional part of the span simplifies them, so that after this substitution they appear in the form of a constant multiplied by the load and the cube of the span.

Continuous-Beam Method—In this case the moments are found for the number of spans over which each band is continuous and which is assumed as carrying one-fourth width of panel loading on either side of the center line. The various moments at the critical points of maximum moment with the condition of loading governing each having been found, these moments may then be reduced in the ratio of the squares of the shortened spans (due to the column flare and capital) to the

spans center to center of columns. For this condition the moments due to dead loads, which are naturally continuous over the multiple spans, and those due to live loads, which require different conditions of loading for maximum requirements, will have various coefficients. The equation is:

$$M = (Kw + K_1 w_1) L^2$$

where K and K_1 are the constants respectively for the dead load w and the live load w_1, causing maximum moments for the given number of spans for the section in question.



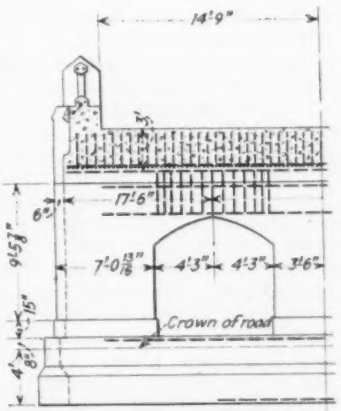
Half Right Section, Center Pier

All the above design methods are approximations. An exact analysis would involve the moments of inertia of the column, flare and drop-panel (where provided), as well as that of the slab, which would result in a very involved set of formulas for determining the moments and shears at the critical sections. The additional factor involved in the application of Poisson's ratio, because of the existence of stresses and deformations at right angles to each other, would still further affect and complicate the problem. The approximate methods give results that check fairly well with actual test results.

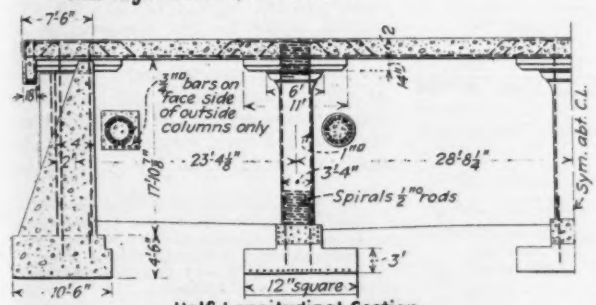
The practice on the Lackawanna Railroad has been to use the continuous-beam method, as most nearly approximating actual conditions of railroad loadings. The procedure is to figure maximum dead-load moments and shears at the various critical sections, place the E-65 engine load in the proper position to obtain maximum moments and shears, apply the full impact factor where high speeds occur or a modified impact for slow speeds, and then to reduce the resulting moments by the application of the shortened span length.

The possibility of the use of the girderless slab for individual crossings came only after analysis showed its adaptability for other than long wide viaducts with multiple equal spans in both directions, providing the continuity that was deemed indispensable. The writer feels that the adoption of this type of design for individual crossings was an important forward step in the progress of reinforced-concrete bridge design.

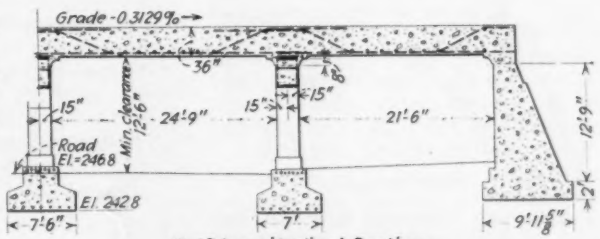
The first individual crossing that was designed as a girderless slab was built in connection with the Orange, N. J. improvements at Central Ave., and served as a direct comparison with a crossing of different design that had been built at Madison, N. J. (Green Ave.-Waverly Place) some years previous. In both cases the bridge was for a 100-ft. street, with supports along the center line and at each curb. Spacing center to center of columns or piers for the roadway spans was about 28 ft., and there was a sidewalk span on either side for the remainder of the street width. The older structure was figured continuous over the three piers and simply supported at each abutment, and



Half Right Section, Center Pier



Half Longitudinal Section
CENTRAL AVE. BRIDGE, ORANGE, N.J.
FOUR-WAY FLAT SLAB



Half Longitudinal Section
GREEN AVE. BRIDGE, MADISON, N.J.
ONE-WAY CONTINUOUS SLAB

Fig. 2—Comparison of a girderless flat-slab bridge (Central Ave., Orange, N. J.) with one using a one-way reinforced slab continuous over three piers and simply supported at the abutments. Note that the comparative slab thicknesses are 24 and 36 in., respectively, for about the same span lengths.

Fixed-Beam Method—In this method each band, either transverse, longitudinal or diagonal, is treated as a beam fixed at the edges of the column capital and loaded with a quarter-panel width of dead and live load on either side of the center line of the band; in the two-way system the additional portion from the middle section is added. This method automatically provides for the shortening of the span length and the resulting reduction of moment. Under this assumption it is evident that only the loading on the band (or panel) under consideration affects the moments in it, the loading of the adjacent or the distant bands not being a factor. On the other hand, this method is predicated on the provision of an unusually stiff column and column capital. Anomalous as it may seem, the longer the span lengths of panels and the heavier the unit loading thereon the more nearly will the requirement of column rigidity be fulfilled; long spans and heavy loading result in large column reactions, requiring large column diameters, thus producing for a given height of column a smaller l/d ratio.

required a slab thickness of 36 in. as a one-way slab (figuring the positive reinforcement continued straight over the supports as compressive reinforcement). The Central Ave. bridge, as a four-way slab, required only a 24-in. thickness of floor depth, thus effecting a saving of 1 ft. of excavation of the 100-ft. street or of a 1-ft. embankment of a three-track railroad roadbed. Another feature in connection with the design of the Central Ave. bridge was the substitution of columns with overhanging slabs and retaining walls for the usual abutments, the columns taking the bridge reactions and the retaining wall the embankment pressures. The drop-panels were chamfered to conform to the clearance requirements and at the face were curved for architectural effect.

Fig. 2 shows the two bridges in section, and a comparison can readily be made. The Central Ave. bridge, in-

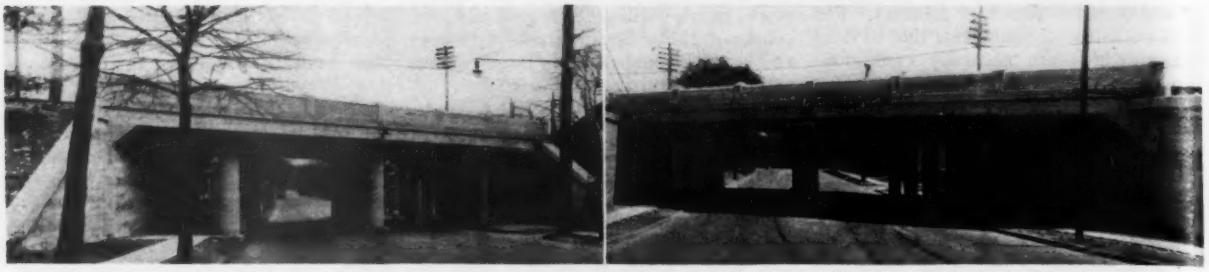


Fig. 3—Girderless flat-slab bridges at Brook Ave. (left) and Main Ave. (right), Passaic, N. J. Both are on about a 45-deg. skew. A 21-in. slab and a 10-in. drop panel are used on both structures, which are built for a 66-ft. width of street.

cidentally, is the only individual bridge on the Lackawanna Railroad having fascia columns; all the other bridges for single crossings have been designed with overhanging parapets.

The successful application of this design led to its use in connection with the East Orange improvements, where four individual bridges were constructed, all differing in their column spacing and their angles of crossing. None of them provided a layout of square or regular panels, so that the use of coefficients would have been of doubtful value. In the design it was necessary to find actual moments at critical sections for each type of loading and to modify these moments for shortening of span lengths and for two-way distribution of the loading as required.

On the bridges built prior to the East Orange improvements, inspection disclosed spalling of the concrete immediately below the bearing, due to the use of a "sliding-type" joint. For this reason the writer decided to design all succeeding structures, including those on the East Orange improvement, for a condition of restraint or partial fixity at the abutments. In the ten years that have elapsed since the latter construction no spalling has been observed.

Skew bridges

The most favorable skew for this type of design is 45 deg., because this permits the use of square panels except at the face, where triangular half-panels result in practically a three-way system of reinforcement. This 45-deg. condition was almost exactly satisfied in the case of two bridges required for bridge replacement in Passaic, N. J., on the Boonton Line (mainly a freight line). The two streets (Main and Brook Aves.) cross the railroad line in opposite directions, forming a right angle at their junction at a point some distance beyond.

With respectively four- and five-track widths of bridges (replacing two-track steel bridges) the four-way slab was found ideally adapted for these crossings. By careful selection of the location of a construction joint it was possible to build each of these bridges in two operations despite the skew. The first section, of sufficient width to accommodate two tracks, was opened to traffic while the remaining section was being completed.

Both of these bridges were built for a 66-ft. width of street, the distance between curbs being set at 0.6 of the total width, or 39½ ft., so that for a 2 ft. 9-in. diameter column there resulted 21¾-ft. transverse span lengths center to center of columns to accommodate the roadways (Fig. 3). A 1 ft. 9-in. thickness of slab with the addition of a 10-in. drop-panel was used for both of these bridges.

Not far from these crossings there was constructed another skew bridge that provided a comparison of the girderless flat-slab bridge of reinforced concrete with a through structural-steel girder bridge of similar length but without columns at the street center lines or at the curbs. This structure was built as a grade-crossing elimination at Broad St., Clifton, N.J. (Fig.1). The street was 70 ft. wide, and the bridge carried a four-track railroad line at an angle of about 40 deg. The roadway spans were made 23 ft. 10 in. c. to c. of columns, normal to the street center line, with 11 ft. 2-in. sidewalk openings on either side to the abutments. The column spacing along the street center line was made 20 ft. c. to c. The slab thickness is 2 ft., with a drop-panel thickness of 11 in. The possibility of comparison with a steel girder bridge arose from the fact that the flat-slab concrete bridge was built after competitive bids by a contractor, who in the course of the same year was the successful bidder for the construction of a double-track plate-girder bridge with girders 93½ ft. over all and a clear span of 86 ft., utilizing a solid concrete waterproofed deck. The concrete structure has a length, measured along the track on the skew, of 108 ft. 4 in. be-

Fig. 4—A girderless flat-slab bridge over Liberty St., Binghamton, N. Y., rests on a foundation mat, also designed as a flat slab.

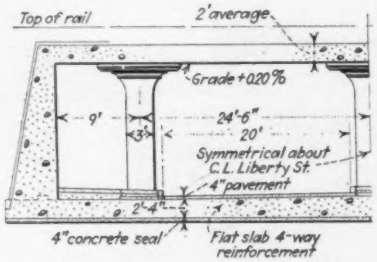


Fig. 5—Partial foundation plan of overhead crossing at Kearny, N. J., showing unusual arrangement of pile footings to accommodate a rectangular column layout for a severely skewed girderless flat-slab bridge.

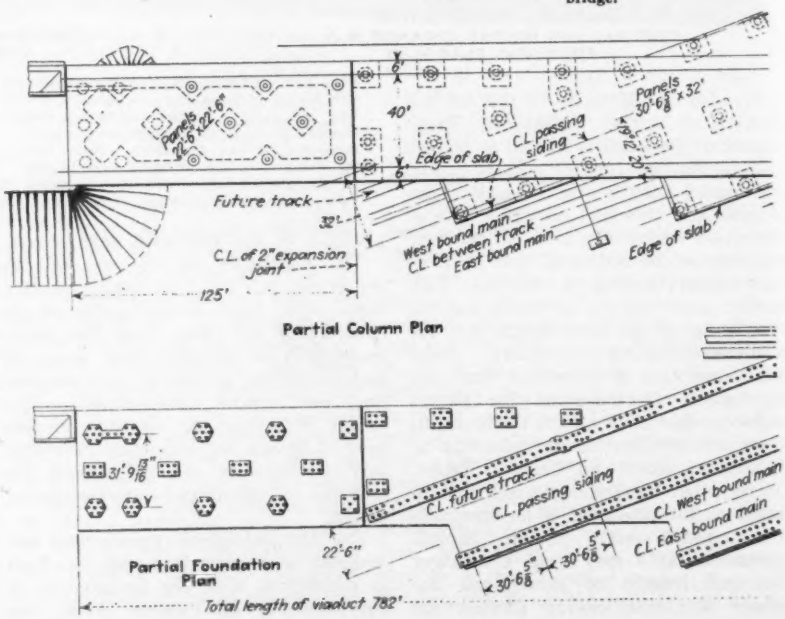


Fig. 6—Girderless flat-slab overhead crossing at Kearny, N. J., the arrangement of which is indicated by the partial foundation plan in Fig. 5.

tween abutments. After making deductions from the cost of the steel structure for deeper foundations and other inequalities, the cost of the two-track steel structure was a considerable percentage greater than that of the four-track concrete structure. This saving is aside from the saving both in embankment for the track and excavation for roadway and street due to the ability to retain the standard 13-ft. track centers that would have had to be 17½ ft. or more for the through-girder construction.

It was for the purpose of great economy that the New York state authorities permitted the adoption of this type of design for the Elmira, N. Y., grade-crossing elimination recently completed, in the course of which five such bridges were constructed, three with and two without drop-panels. These bridges which were described in the *Engineering News-Record* (Dec. 8, 1933, p. 678) are all skew bridges with irregular panels.

Economy also dictated the use of girderless flat-slab bridges for a series of grade eliminations at Binghamton, N. Y., in which the Erie and the Delaware & Hudson railroads are also involved. On Liberty St., for example, four bridges of this type have been constructed, two by the Lackawanna and one each by the other roads. The Lackawanna and Erie bridges were designed with a girderless flat slab for the foundation mat as well as for the deck, providing additional ecenomy. The Lackawanna bridges were designed for an E-65 engine loading, with full impact allowance, while an E-70 loading was used on the Erie bridge.

Carrying highways over railroads

As illustrating the adaptability of the girderless flat slab, there have been built two overhead structures of this type along the lines of the Lackawanna. The first of these was installed in 1919 at Kearny, N. J., as a grade-crossing elimination of a country road known as Sanford's Crossing. It presented great difficulties in design because of (1) an unusually large angle of skew, (2) the necessity for providing for six tracks because of siding facilities for possible industrial development of the Hackensack Meadows, (3) the sharp change in span lengths and column spacing in the approaches, and (4) the soil conditions available for foundations, which required the use of piles. The partial plan, Fig. 5, shows the center section over the tracks with 32-ft.-square panels requiring a 1 ft. 3-in. slab thickness as well as an 8-in. drop-panel at the columns, and an approach section having 22½-ft. square panels, with diagonal column arrangement, requiring 10½-in. thickness of slab, with drop-panels throughout. The columns in this structure are of octagonal cross-section, which may yield a better appearance but cost too much.

A similar plan was adopted for a grade-crossing elimination at Kensington Ave., Buffalo.

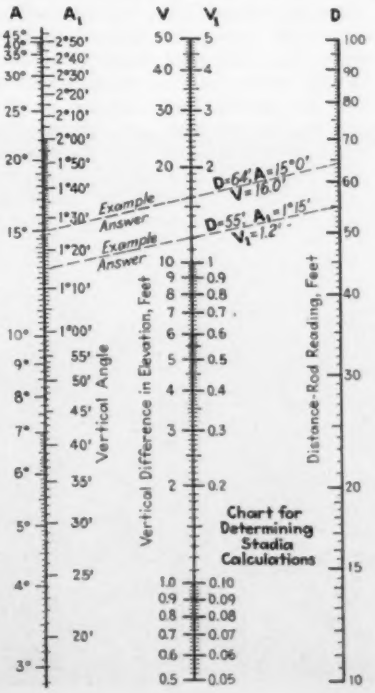
All these girderless flat-slab structures were designed with allowable unit stresses of 650 ft. per sq.in. for the concrete at the center and 750 lb. per sq. in. at the supports. Reinforcing-steel stress was 16,000 lb. per sq.in.

Vertical Distances Charted From Stadia Measurements

By J. R. McComas

Assistant Sanitary Engineer, Maryland Department of Health, Baltimore, Md.

THE accompanying chart showing the vertical differences in elevations corresponding to any vertical angle and any stadia distance should be found helpful to the field engineer, especially in preliminary location survey work. Generally several readings suffice to determine the maximum existing difference in elevation of a plat of ground. When such readings are made by the stadia method it becomes necessary to convert the vertical angle and sloping distance readings into actual vertical differences and actual horizontal distances. The latter, in the majority of cases, is of minor importance and for the purpose at hand is not considered.



Stadia data, so obtained, are generally converted by the use of a special type of slide rule, which is not a desirable piece of field equipment. As a substitute, the accompanying chart may be pasted in the field notebook and used for this purpose. Sufficiently accurate results may be obtained as are required for field determinations.

The distance axis D is graduated from 10 to 100 but may be used for multiples thereof by proportioning the values found on the V axis accordingly.

Two examples are indicated to bring out the purpose of the double scales on the A and V axis. In every case, irrespective of the distance reading, use A with V only and A_1 with V_1 only.

Example 1—Given a stadia distance reading of 64 ft. and a vertical angle of 15 deg., connect, with a straight line, 64 on the D axis with 15 deg. on the A axis and read 16.0 on the V axis. The answer is 16 ft.

Example 2—Distance, 55 ft., vertical angle, 1 deg., 15 min. Connect D with A_1 and read 1.2 ft. on V_1.

If, in example 1, the distance had been 640 ft., the vertical difference in elevation, read from the V axis, would be 160 ft. instead of 16.0 ft.

Third of Four Articles on the Design and Construction of Rolled-Earth Dams

Field and Laboratory Verification of Soil Suitability

Consolidation and percolation tests and test results —Saturated plasticity—Swelling and bearing power

By R. R. Proctor
Field Engineer, Bureau of Waterworks and Supply, Los Angeles, Calif.

IT IS advisable to determine the dry weight, plasticity and watertightness of a soil when under a load corresponding to the weight of soil and water that will be placed above it in the finished dam. The determination of the percolation rate of a compacted soil not under load is of little value, except to determine the relative watertightness of several soils; this comparison is best made when the soils are under a load equivalent to about 10 ft. of fill. As all soils consolidate somewhat when placed under load, it was decided to measure the soil consolidation and saturated plasticity with the same test equipment that is used for the percolation rate.

A cylinder about 8 in. in diameter, which is closed at the bottom except for a water inlet, is used. A rigid but porous disk is placed in the bottom to insure even distribution of the water for the percolation test. A little more than 3 in. of compacted soil is placed on top of the disk. It is compacted in the standard manner and at the moisture content shown by the compaction curve to result in the desired dry weight. Soil removed from a dam may be placed at the dry weight of removal in this manner. The top of the compacted soil is then cut away until a depth of 3 in. remains. The weight and volume of the compacted soil is used to check the compacted dry weight. Another porous disk similar to the first is placed on top of the soil. A loose-fitting plunger is then placed over the disk. It has projections that extend over and beyond the edges of the cylinder at opposite sides, so that a dial gage, reading to one thousandth part of an inch, may be fitted between the projections and the base plate, permitting the accurate measurement of any settlement or consolidation that occurs. A coil spring of the proper capacity to give ample travel for the load desired is placed on the plunger and is compressed with a plate on its top, which is held to the base plate by nuts on long threaded rods.

Any desired load may be placed on the spring and soil in a compression machine, after which the nuts are tightened sufficiently to relieve the machine of the load. The spring then continues to exert this force for as long a time as desired, if the soil consolidation is not enough to enable the spring to stretch enough to reduce the load materially. The load on the spring may be verified at any time by placing the apparatus in the compression machine and noting the pressure required to relieve the tension in the rods sufficiently for the nuts to be turned with the fingers. The entire assembly is shown in Fig. 2.

Fig. 1—The upstream face and left abutment of Bouquet Dam No. 1 of the bureau of water works and supply of the city of Los Angeles, showing the compacting of the earthfill and preparations for placing the reinforced-concrete apron.

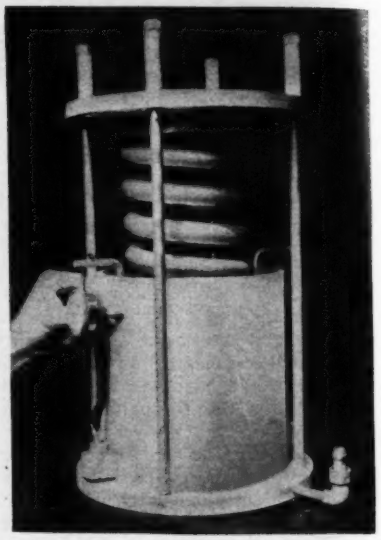


Fig. 2—The consolidation-percolation cylinder, showing the methods of measuring settlement, applying load, and the water inlet for the percolation test.

The compacted-soil specimen may be subjected to any desired load and the resulting settlement or consolidation observed until it ceases. The load may then be increased and additional consolidation data obtained. The change in dry weight, which will take place in any part of the dam during the construction period due to consolidation caused by the weight of the soil to be placed above, may be determined by this test. Fig. 4 shows the consolidation that resulted after compacting the sandy clay soil of Fig. 7 (*ENR*, Sept. 7, 1933, p. 287), to four dry weights and placing it under loads equivalent to the depth of fills shown. These results indicate that a depth of fill of at least 200 ft. is required to correct fully, by soil consolidation, an error of 6 to 8 lb. in the dry weight of placing.

Plasticity and percolation

The plasticity-needle reading of the soil at the time of placing in the cylinder may be determined after compacting a portion of the soil, as prepared for the consolidation cylinder, in the compaction cylinder. The plasticity-needle reading of the consolidated soil may be taken upon removal of the load. This value, taken immediately after the consolidating load is released and after the completion of the percolation test, represents the plasticity-needle penetration resistance of the soil in place after the dam is completed and in service. It

therefore determines the extreme softening that can take place in the dam at the point from which the soil sample was taken.

The percolation rate is measured, after the consolidation is complete and with the soil still under pressure, until a constant rate of flow is obtained. The quantity of water required to maintain the desired pressure head is measured. A pressure head of about 12 ft. is usually used in order to determine the results in a few days. The water pressure acts upward, therefore the soil must be under a downward load greater than the upward force exerted by the water, equivalent to about 5 ft. of compacted soil in this case. Consolidation-percolation tests under less load require lower heads and very much longer time to complete.

It is best to determine the percolation rates of all soils under a load equal to that of 10 ft. of fill. The applied downward load is then about twice the upward water pressure, which insures the soil specimen from being moved. Many well-compacted soils require only a few drops of water to be added each day to maintain a 12-ft. head on a 3-in. compacted-soil layer. Most compacted soils have a percolation rate that is very much less than the surface evaporation; for this reason it is best to place enough water on top of the specimen to cover it and prevent surface drying. The rate of flow is reduced by direct-head ratio to that at which the pressure head over the water standing on the top of the specimen equals the thickness of the soil sample. This is a hydraulic gradient of 1. The percolation rate is determined by dividing the total quantity of water that would flow through a soil specimen in a year at a hydraulic gradient of 1 by the area of the specimen through which the water passed. It is expressed in feet per year.

The percolation rate expressed in this way is readily adapted to computing flows through dams and thin layers of compacted soil; it is not necessary to compute actual velocities through the voids. If the area exposed is one acre and the percolation rate is 1 ft. per year with a hydraulic gradient of 1, the total yearly flow is 1 acre-ft. of water. This is 0.00274 acre-ft. per day, 0.00138 sec.-ft., 0.62 gal. per minute, or 893 gal. per day. A dam having a height of 200 ft. and an average length of 650 ft. has a sectional area of 3 acres exposed to water flow at a hydraulic gradient of not more than 1. If it were constructed of compacted soil with a percolation rate of 1 ft. per year, it would have a maximum leakage of 1.86 gal. of water per minute. This quantity of leakage is extremely small for such a dam.

The percolation rates determined from nine soil samples similar to that of Fig. 5 (*ENR*, Sept. 7, 1933, p. 287) averaged 0.04 ft. per year at an average dry weight of 126 lb., resulting from consolidation at loads representing 75 to 180 ft. of fill. The maximum rate was 0.120 ft. per year; the minimum was 0.001 ft. per year. A dam of the dimensions previously mentioned would then pass not more than 1/25 of 1.86 gal., or 1/15 gal. per minute, if constructed throughout of soils similar to the nine samples. Such a dam might be classed as watertight. It would take many years for the water to flow through. The consolidated soils, when fully saturated, would have a plasticity-needle reading in exces of 1,000 lb. per sq.in., a large factor of safety against failure by softening from saturation.

The accompanying table shows the results of 27 consolidation and percolation tests on several types of soils when under loads equivalent to fills having depths of 2 to 200 ft. The dry weights, plasticity-needle penetration resistances and moisture contents are given for the following conditions: the peak of the dry-weight curve obtained by the standard laboratory compaction method, the conditions of placing in the consolidation cylinder, and the conditions found in the compaction cylinder after the load was released and the specimen exposed at its top. The percolation rate through the specimens when they were under the various loads is shown. The percentage of the weight of the soil in the specimen that could be washed through the 200-mesh screen, 0.074-mm. clear opening, is shown.

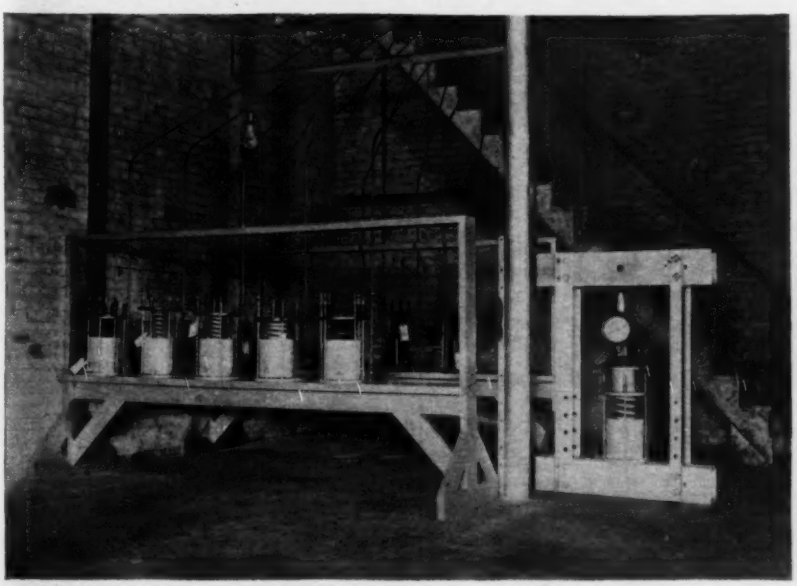


Fig. 3—Laboratory equipment for consolidation and percolation tests. The method of applying equivalent fill loads is shown at the right.

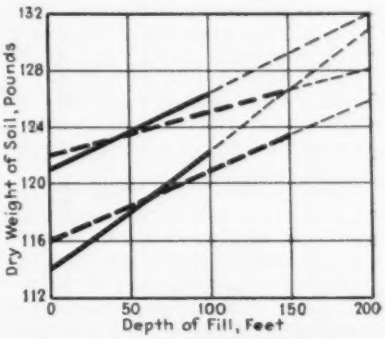


Fig. 4—Curves showing the increase in dry weight of a sandy clay soil caused by soil consolidation, under pressures equivalent to various depths of fill.

No swelling from saturation

Nine of the soil specimens were placed under 10 ft. of fill load or less. All had a dry weight, after consolidation and saturation by percolation, in excess of that to which they were compacted in the test apparatus. It may therefore be assumed that these soils will retain their watertightness and stability in a dam; they will not swell when saturated, thereby decreasing in watertightness and stability. The light load applied forced the specimens to consolidate; if any force of swelling existed in sufficient amount to exceed the weight of the indicated depths of fill, it would have forced the spring upward, permitting the specimen to swell. If a soil should be found in which this occurs, it is a simple matter to increase the load on it until such a force, if it exists, is overcome. The extent of any swelling at any intermediate load could easily be determined. Further tests on very fine soils may indicate this tendency, although no soil has been found to swell at this time.

Importance of compaction

An inspection of the percolation rates and the percentage of the soils finer than the 200-mesh screen shows that no definite relation exists between them. It is shown, however, that the extent to which the soil is compacted is much more important for obtaining watertight-

ness than the percentage of fines. Specimen No. 17 is an interesting example. This soil, containing 8 per cent finer than the 200-mesh screen, appears to be a gravel. When the rock above ¼-in. size is removed, the remainder appears to be a sand suitable for low-grade concrete. It has none of the characteristic feel of a clayey sand; it cannot be compacted in the hand to a firm ball. This soil was very watertight after consolidating under a 50-ft. fill load to a dry weight 6 lb. more than the maximum of the laboratory compaction method. This shows the possibility of compacting soils very low in fines to a condition suitable for dam construction.

Specimen No. 3 shows the effect of saturation on a soil that is insufficiently compacted at a low moisture content. The dry weight of the soil as it was placed in the consolidation cylinder was 19 lb. below that obtained by the laboratory compaction method with a moisture content 2.8 per cent higher. The plasticity-needle reading was 1,250 lb. per sq.in. When the soil was saturated under a 4-ft. fill load, the plasticity-needle reading decreased to 90 lb. per sq.in. This increased plasticity was caused by the increase in moisture content from 15.6 to 30.2 per cent. The dry weight of the specimen increased from 89 to 92 lb. This relatively low dry weight accounts for the high percolation rate. The percolation rate of this soil, if compacted to a dry weight of about 108 lb., would probably be not more than 0.010 ft. per year. It is interesting to note that the percolation rate of this clayey soil as it was compacted is many times greater than the very sandy specimen previously discussed.

A great many comparisons can be made from the accompanying table. There are many variations in placing and loading conditions. None of the soils show an excessive percolation rate, and most of the rates shown are so low that the soil may well be classed as impervious to water. The percolation rates through all soils that have been tested are negligible when they are compacted to the condition of a minimum plasticity-needle penetration resistance of 300 lb. per sq.in. when saturated. Any soil that meets this condition could probably be used in a dam; it is unlikely that the percolation velocity would be sufficiently high to erode any of the fines from the soil.

The minimum dry weight to which a soil must be compacted is easily found by the consolidation-percolation test. It is only necessary to compact the soil to lower dry weights and observe the results of the test. The lower limit of the dry weight to which the soil must be compacted is determined when the minimum allowable saturated plasticity-needle pentration resistance or the maximum allowable percolation rate is reached.

Testing foundation soils

Earthfill dams are often desirable at locations where the cost of stripping the foundation to bedrock prohibits their construction. The soils that are to be the foundation for an earth dam may be subjected to the same analysis as a compacted fill. The watertightness and stability of such soils is of equal importance to those used in the fill, particularly on the abutments, where the consolidation resulting from the lesser depth of fill is not so great.

The dry weight of foundation soils is easily determined at any depth by excavating a shaft or side-hill cut to sufficient dimensions to permit the measurement of the dry weight in place, as outlined for compacted fills. The soil may then be tested for consolidation and percolation after being placed in the test apparatus at the dry weight in place and under such load as the contemplated dam will produce at the point of sampling. The stability of the soil under these conditions can be determined by use of the plasticity needle at the conclusion of the test. The probable settlement of the foundation soils can be determined very closely by a series of tests of this nature.

It is necessary carefully to consider the rate at which the settlement takes place in the consolidation cylinder. A slow settlement of the small specimen indicates that the total foundation settlement may not take place for considerable time after the dam is completed. If this settlement can occur only by forcing some of the water from the soil, it is possible for the weight of the dam to force the saturated and quite plastic foundation soil to move outward, causing at least a partial failure of the dam before the full consolidation has taken place.

Bearing power of saturated soils

The capacity for water absorption in a soil has been shown to be determined by its dry weight. The soil used in the preparation of Fig. 7 (*ENR*, Sept. 7, p. 287) can be found in place at a dry weight of 110 lb. with a moisture content as low as 6 per cent. This soil has a plasticity-needle penetration resistance of about 10,000 lb. per sq.in. in this condition. The moisture content for complete saturation at this dry weight is 20 per cent, and the plasticity-needle penetration resistance then decreases to 20 lb. per sq.in. A standard bearing test on this soil in the drier condition would indicate a capacity to support a high unit load, and a similar test on the soil in its saturated condition would indicate a capacity to support only a very light unit load.

The determination of the dry weight

RESULTS OF CONSOLIDATION AND PERCOLATION TESTS OF VARIOUS SOILS

	Soil Characteristics at Maximum Density Obtained by Standard Laboratory Compaction			Soil Characteristics as Placed in Consolidation Cylinder			Soil Characteristics as Determined After Saturation by Percolation-Consolidating Load Released					
Test No.	Dry Weight, Lb.	Plasticity-Needle Reading, Lb. per Sq. In.	Moisture Content, Per Cent	Dry Weight, Lb.	Plasticity-Needle Reading, Lb. per Sq. In.	Moisture Content, Per Cent	Consolidating Load Equivalent Depth of Fill, Ft.	Dry Weight, Lb.	Plasticity-Needle Reading, Lb. per Sq. In.	Moisture Content, Per Cent	Percolation Rate, Gradient of 1 Ft. /Year	Soil Passing 200- Mesh Screen, Per Cent
1	121	1,200	13.0	111	40	20.2	2	118	75	17.0	0.0710	21.7
2	112	500	15.5	109	425	16.8	3	111	400	19.3	0.0154	51.2
3	108	425	18.4	89	1,250	15.6	4	92	90	30.2	2.000	68.0
4	116	1,150	14.0	116	450	15.6	5	118	475	15.8	0.0154	41.0
5	110	850	13.5	117	1,500	14.2	10	118	1,250	15.0	0.790	29.5
6	124	3,000	11.5	123	500	14.5	10	125	1,200	13.5	0.4700	18.6
7	109	525	18.5	108	300	18.8	10	111	460	19.1	0.0011	52.7
8	122	1,400	12.5	125	1,550	11.8	10	127	1,350	12.5	0.0047	34.2
9	112	825	14.8	114	550	16.4	10	115	550	17.5	0.0028	33.4
10	102	650	22.5	82	5	36.0	18	88	50	31.8	0.1090	38.0
11	89	430	27.5	91	250	32.5	20	93	300	31.6	0.00016	82.0
12	120	950	13.0	117	375	15.0	20	123	450	14.5	0.0035	37.6
13	113	650	16.5	111	425	17.9	25	112	575	18.5	0.0008	50.6
14	115	750	15.2	111	375	17.5	30	113	400	17.6	0.0010	46.2
15	110	650	18.2	111	550	17.2	35	118	900	16.4	0.0014	39.6
16	102	650	22.5	82	5	36.0	45	93	120	29.3	0.0280	43.0
17	121	700	12.5	117	475	15.2	50	127	950	12.9	0.0156	8.0
18	123	1,000	12.0	118	450	14.2	50	125	900	13.5	0.0007	38.3
19	128	1,450	11.2	126	1,225	11.2	50	131	1,800	11.2	0.0050	29.4
20	128	2,500	11.0	122	500	12.8	50	128	1,300	12.6	0.0096	31.7
21	124	1,500	12.0	114	500	13.5	100	119	700	14.7	0.8000	30.6
22	124	1,500	12.0	121	600	13.5	100	125	1,410	14.4	0.0100	30.6
23	123	2,000	11.4	122	675	13.5	140	126	700	12.4	0.0018	37.0
24	122	1,100	12.0	114	1,130	11.7	150	121	500	14.8	0.0450	32.4
25	122	800	13.5	123	1,050	12.8	150	127	1,700	12.4	0.0167	36.4
26	123	1,200	12.0	120	350	15.0	180	124	1,800	13.1	0.0010	46.4
27	123	750	12.5	119	450	14.7	200	127	1,700	12.4	0.0450	35.2

of a soil in place can therefore furnish data of much value in determining the bearing power of soils, particularly if the moisture content of the soil is low but is subject to considerable increase after the load is applied. The soil can be placed in the consolidation-percolation cylinder at the dry weight in place, and a load can be applied equal to the contemplated foundation pressure. The observed settlement would be that to be expected immediately under the foundation. The application of other lighter loads determines the settlement to be expected at various depths below the foundation, depending on the manner in which the foundation pressure is assumed to be spread in the soil below. The plasticity needle can be used directly on foundation soils in place if they are known to be saturated. Further study and experiments will undoubtedly establish a relation between the bearing power of a soil and its plasticity-needle penetration resistance when saturated.

Required depth of stripping

The dry weight of natural soils in place has been found to increase as much as 10 lb. at a depth of 15 ft. below the surface. A soil that was wholly unsuitable for foundation purposes at the surface had a dry weight equal to that to which it was necessary to compact it in the dam at depths of 12 ft. below the surface. The depth to which foundation soils should be stripped is that to which the dry weight in place is the same as required if the soil were to be placed in the fill. If the foundation soils are too deep for this to be accomplished, the cross-section of the dam should be carefully designed to meet the conditions of stability that are thus imposed. The compacted fill should then be placed at the same saturated plasticity as the foundation in order to result in a fill of sufficient plasticity to conform to such settlement as will take place in the foundation. The careful use of the principles that have been set forth should permit the design and construction of compacted-earth dams of known safety at locations that have been considered unsuitable for this purpose.

Stability of existing dams

The principles that have been set forth may be used in an analysis of the safety of existing earthfill dams. The determination of the dry weight of fill and foundation soils at critical points and the preparation of the soil-characteristic curves, similar to Fig. 7 (*ENR*, Sept. 7, p. 287), from the removed samples can result in a definite conclusion as to the margin of safety which exists in the structure. The determination of the dry weight in place for soils below the line of saturation in an earth dam is easily made. A sample of the saturated soil is removed by means of a core barrel so arranged that it will bring the sample to the surface in its original condition. The determination of the moisture content and specific gravity of the saturated soil determines its dry weight, as shown in Fig. 7 (*ENR*, Sept. 7, 1933, p. 287). The removel of sufficient soil for the compaction and plasticity-needle test enables the preparation of curves similar to those of Fig. 7, in the previous article. Sufficient information is now at hand to proceed with the consolidation and percolation tests that have been described. This information is of particular value when the problem of increasing the height of an existing dam is being studied.

Paving Side Ditches for Hilly Roads

Cheap stone paving developed from trial by maintenance department of Indiana state highway commission

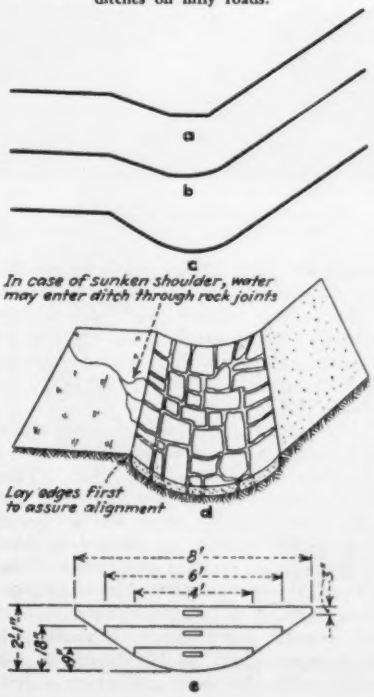
By Don M. Norton
Superintendent of Maintenance, Seymour District, Columbus, Ind.

DURING the past three years the maintenance forces of the Indiana state highway commission in the Columbus subdistrict have been experimenting on types of stone side-ditch paving, to produce some means of caring for ditches on roads through hilly country where there are miles of ditches, which because of soil conditions are a source of continual repair and expense. A ditch cross-section and method of laying stone have been developed and have proved to be very satisfactory and inexpensive.

This type of ditch paving does not answer all purposes, nor does it take the place of concrete paving, which is necessary on very steep grades; but it does have one advantage over the concrete ditch in that it allows water to enter from the road shoulder at each joint and eliminates the false ditch ordinarily found along the ditches paved with concrete. The cost of the stone ditch is of course another thing in its favor.

Fig. 2—Stone-paved side ditch on Indiana highway.

Fig. 1—Diagram sections, templets and pavement details of stone paving for side ditches on hilly roads.



When this work was first started, the stone was laid in ditches with a cross-section which is common in all road construction (Fig. 1*a*). This was found to be very tedious and not at all satisfactory. Later the cross-section was changed to eliminate the sharper angle (Fig. 1*b*), which helped some but did not yet fit our needs. At present the ditches are being laid to a cross-section (Fig. 1*c*) that eliminates all angles, making the work much easier for the workmen and giving each stone an equal bearing on its neighbor throughout the ditch. This also reduces the possibility of misplacement by water and frost.

Fig. 1*e* shows templets used for ditches of different widths from 4 to 8 ft., depending on the amount of water to be carried. The ditch is first formed to fit the templet, then flat stones of near the same thickness (2½ to 4 in.) are laid, starting at each outer edge to insure a good line both at the road shoulder and toe of the back slope and finishing in the center of the ditch. Very

little skill is required in laying the stone in this manner, and no grout or mortar is necessary.

The accompanying view (Fig. 2) shows a ditch constructed along Indiana state road No. 7. These ditches are in perfect condition after having withstood the winter and are good for many years of service without any further attention.

The cost of this work depends entirely upon the condition of the ditches and the distance that the material has to be hauled. The work shown followed close after the road was constructed, with ditches in good shape, the stone costing an average of $1.25 per cubic yard delivered. More than 3,000 ft. of paving was laid an average of 5 ft. wide, which cost $0.16 per linear foot.

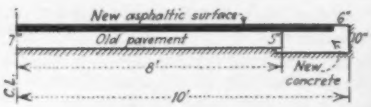
Low-Cost Resurfacing of Old Pavement

Old concrete road widened 4 ft. by concrete strips and surfaced with asphaltic concrete at cost of 40c. a square yard for surfacing

By John E. Blair
Division Engineer, Texas State Highway Department, Austin, Tex.

RESURFACING old cement concrete roads that were built several years ago under different traffic conditions has become an important matter in many parts of the country. In a report of the Highway Research Board, at its 1932 annual meeting, reference is made to costs in different parts of the country using asphalt mixtures that ran from $5 to $12 per ton, and it is in this connection that some comment may be interesting as to the same character of work recently accomplished in Texas at materially lower prices.

An old concrete road, 16 ft. wide, 5:7:5 in. in cross-section, was widened to 20 ft. and then resurfaced with an



Half section of widened and resurfaced concrete pavement. Note narrow curb on widening strip.

asphaltic mix. The old concrete was very rough, having been laid on an unstable black waxy soil, and it was some ten years old. To correct the crown and the rough surface, the variations in the depth of the new asphalt surface were from 1 in. minimum to as much as 5 in. maximum at the deepest point. The accompanying sketch shows the method of widening and the type of curb used to correct the steep crown. This curb was built with a narrow raised edge in place of the 2-ft. concrete edge, because of the better appearance of the narrow concrete edging and because of the greater invitation for lateral dispersion of traffic when the same type of surface extends for the full width of the pavement.

The old surface was cleaned by sweeping, after which it was primed with a medium-curing cutback asphalt of low viscosity. The rate of application was 1/10 gal. per square yard, and it was thoroughly worked into the surface by scrubbing with brooms. This priming operation penetrated all of the little crevices in the old surface, tending to waterproof against groundwater and also to form a thorough bond between the new surface and the old. On the surface so prepared an asphalt-stone mix was spread and screeded with a hand-drawn screed that operated over the concrete curbs and struck off the mixture to a 1-in. crown. From experience obtained in the first section of the work, it is believed that this screeding operation is unnecessary and that by careful

Old concrete road and widened and resurfaced section shown in contrast.

operation of a blade-grader an equally smooth riding surface can be obtained. The surface should be checked frequently with a templet, and there is no reason why a finished surface having variations of less than ¼ in. in 10 ft. should not be obtained.

For mixing the stone and asphalt, a 7-ft.-capacity standard concrete mixer, mounted on a truck, was used. This truck outfit was driven alongside the cars containing the stone and produced the mixture so as to permit gravity flow of the finished product into trucks. The stone was conveyed to the mixer by four men counting shovelfuls, and the asphalt was poured with a bucket. About ½ gal. of kerosene per batch, placed in the mixer with the stone and given a few

Asphalt surfacing mixed in concrete mixer on truck charged by hand and discharging by chute into truck.

turns before the asphalt was introduced, markedly improved and expedited the mixing. The size of the stone used was 100 per cent passing the ¾-in. screen and 100 per cent retained on the ¼-in. screen. Approximately 8 gal. of asphaltic material was used with each ton of stone. This produced a thoroughly coated aggregate, but one of very open character. After the mixture had been bladed for several hours, it reached a tacky condition and was then thoroughly rolled. Any low places that developed under the roller were corrected by additional mixture until no further settlement occurred and a uniformly smooth surface had been obtained. Approximately one week later a very light seal coat was given, using the same cutback asphalt covered with a light application of coarse sand, followed by final blading and rolling. The cost of the stone was $2 per ton f.o.b. siding, about one-half mile distant from the work proper. The cost of the asphalt f.o.b. siding was 4½c. a gallon. An average thickness of surface of 2 in. was obtained at a total cost of 40c. per square yard.

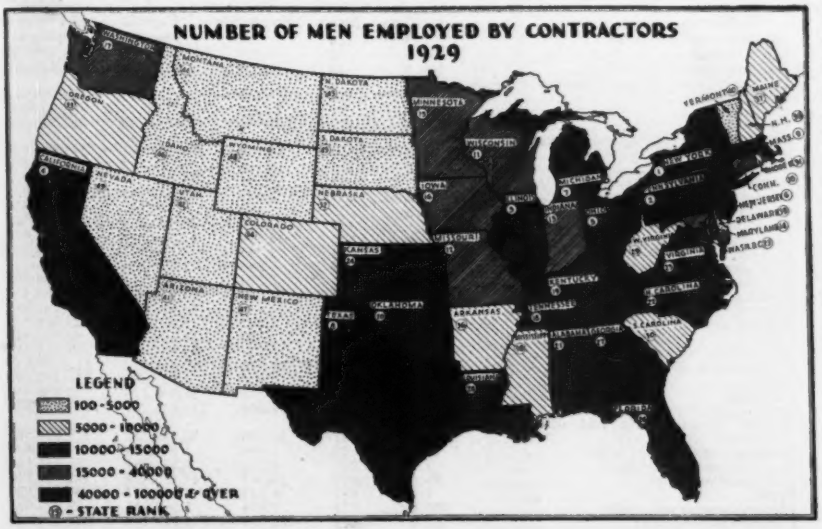
This work was somewhat in the nature of an experiment, and consequently hand methods were employed to a considerable extent. It is believed that the cost can be reduced somewhat by the use of a larger mixer and the handling of aggregates by clam-shell bucket to storage bin above the mixer. The priming of the base could be carried on with pressure distributors. All of the work was carried on under traffic with relatively little inconvenience.

Employment and Values in Construction

A circumstantial survey of the construction industry, tabulating employment and values in totals and subdivided for raw-materials production, manufactures, transportation and distributor agencies

By Lawrence Mehren
Construction League of the United States, Washington, D. C.

THE INFLUENCE of the far-flung and complex construction industry, its importance as an employment agency, the penetration of its various functions and divisions into every section and corner of the country and its effects on other industries have been the subjects of a survey just completed by the Construction League of the United States. Data from scattered sources have been assembled and analyzed and the results pieced together to reveal a startling picture of an industry second only to agriculture in national economic importance. Selling facts have been disclosed, revealing that a tenth of America's workers depend on construction for their livelihood, that every man employed on the construction site is matched by another in factory, office, mine or on the railroads, and that construction is cradled in sections and towns far from the site of structure or project.



Graph showing numbers employed by contractors in construction based on census figures for 1929 and relative volume of employment by states.

Reason for survey

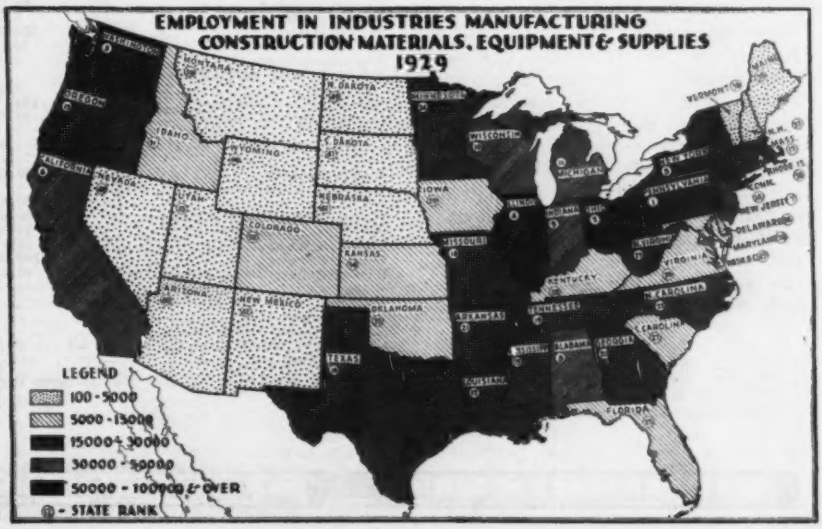
Figures determined in this survey are based on the year 1929; this year was selected principally because its records are far more complete than those for preceding or following years. The actual figures, then, are several years old, but they are hardly more than necessary incidentals, for volume indications are not, in the last analysis, the end of this survey.

To know that the production of construction materials employed every tenth manufacturing worker, that this fabrication went on in a quarter of the country's factories and mills, that one of every five carloads of freight moved carried construction materials, that 8 per cent of the wholesalers of the country were busied with the distribution of construction goods, that 5 per cent of all coal mined was burned to produce these materials, that many industries lean heavily on construction for their existence, and that the multiple division of the construction dollar proves it to be a tremendous factor in our complicated and interlocking economic life—to discover these and other similar indications, which are fairly stable from year to year, is the purpose of the study, and actual volume figures are no more than a means to this end.

The construction industry has long been in need of some accurate appraisal of itself to awaken the industry and the country to a consciousness of its importance and magnitude. The construction interests have never been able to present accurate figures to governmental and commercial agencies to convince them of construction's true place in our economic life. Accordingly, that construction might take its rightful place as the second largest industry and that some calibration might be available to assure careful industrial planning and a dovetailing with our general economic program, the Construction League initiated this survey.

In order that a full realization of the importance of construction would be possible, the study attempts to measure size, divisions, distribution, relation to other industries, employment furnished, value of products and payroll for each industry which serves construction directly or indirectly.

Construction furnishes employment not only on the project site itself but



Graph of numbers employed in industries manufacturing materials, equipment and supplies used in construction, based on census figures and other official statistics for 1929.

in the factories that supply its materials, in the mines, quarries and lumber camps that send crude goods to these manufacturing establishments, on the railroads that transport raw and finished materials, in the coal mines where fuel is produced to make possible the fabrication, and in offices and agencies that distribute the materials.

Actual construction work is necessarily more intense in big cities and industrial areas, but the tributaries to the industry are widely dispersed and penetrate to every corner of the country. The raw materials, manufacturing and mining establishments that are the backbone of construction are spread from coast to coast, and many a small town draws its life blood from supplying the goods which make possible the erection of structures in distant places. Concretely, the Boulder Canyon project has already drawn materials from factories in thirteen scattered states, which in turn obtain their raw materials from other distant places.

Divisions of survey

The survey of the construction industry, then, falls into six general divisions: (1) production of raw materials and semi-manufactures; (2) manufacture of materials, equipment and supplies; (3) transportation of these goods; (4) distribution of the materials by wholesale and retail agencies; (5) design by architects and engineers; (6) the field forces actually engaged in construction work. A study of coal consumption by the raw materials and manufacturing industries has also been included, so that a full realization of the indirect labor attendant to construction might be possible.

All the data are from government sources and every effort has been made to insure accuracy and completeness. Wherever it was necessary to make estimates, either trade associations or government experts have been asked to supply figures covering their particular fields. Detail records of the Bureau of the Census in Washington played an important part in this study, and the records of the Interstate Commerce Commission, the Federal Employment Stabilization Board, and several private agencies have been drawn upon.

The general summary of the survey (Table I) reveals 4,500,000 men serving construction directly or indirectly in 1929—a tenth of gainful employment in all occupations in the country. Direct employment on the construction site accounted for 2,400,000 men, a little more than half of the employees,

One out of every five carloads of freight transported in 1929 consisted of construction materials in raw or finished form.

TABLE I—GENERAL SUMMARY. SURVEY OF THE CONSTRUCTION INDUSTRY, 1929

Division	No. Employed	Payroll	Value*	Per Cent by Emp
1. Raw materials and semi-manufactures..	89,973	$133,316,075	$422,527,097	2.1
2. Manufacture of materials, equip. and suppl	1,097,117	$1,580,484,876	$5,532,590,213	....
Prod. of coal for manuf. and raw mater..	24,153	31,928,384	50,851,019	....
Division 2 totals	1,121,270	1,612,413,260	5,583,441,232	25.3
3. Transportation of raw and finished materials by Class I railroads	183,287	353,688,883	635,671,832	4.1
4. Distribution of mater.:				
Wholesale	137,706	286,539,717	4,050,592,185	....
Retail	309,618	390,386,524	2,827,671,256	....
Division 4 totals	447,324	676,926,241	6,878,263,441	10.1
5. Design of structures and projects (prof. service)	144,661	Not available	11,000,000,000	3.2
6. Construction:				
Contract	1,252,446	2,233,646,882	5,759,774,790	....
Railroad	367,472	391,316,419	1,370,000,000	....
Other (done directly by govern., pub. utilities, power co., etc.	820,591	1,487,731,483	3,870,225,210	....
Division 6 totals	2,440,529	4,112,694,784	11,000,000,000	55.2
Grand Total Construction	4,426,944	6,889,039,243	....	100.0
Total gainfully employed all occupations...	48,829,920	(Therefore direct and indirect employment for construction is 9.1% of all)		

*Value: for division 1 refers to net value of products; for division 2 refers to gross value of products; for division 3 refers to freight revenue; for division 4 refers to net sales; for division 5 refers to value of work designed; for division 6 refers to value of construction work.
(For sources see tables for each division.)

TABLE II — MANUFACTURE OF MATERIALS, EQUIPMENT AND SUPPLIES FOR CONSTRUCTION, 1929

N. B. All columns except "number of establishments" represent figures attributable to construction materials equipment and supplies manufactured.)

Industry	Per Cent of Ind. for Cons.	No. Estab.	No. Employees	Payroll	Value of Products
Aluminum manufactures	0.3	150	71	$106,326	$385,553
Asbestos products	18.5	67	1,709	2,570,829	10,399,704
Asph.-felt-base floor cover	100.0	14	2,754	4,625,295	11,888,962
Bolts, nuts, washers, rivets, not made in rolling mills	50.0	117	9,113	14,532,413	51,196,050
Cast iron pipe	80.0	72	16,829	21,042,124	64,208,440
Cement	100.0	174	38,756	62,834,198	267,509,260
Clay products (brick, tile, sanitary ware, etc.)	60.6	2,062	85,230	109,578,071	241,695,523
Concrete products	94.0	2,438	20,395	29,175,218	87,345,122
Copper, tin, and other sheeting	90	2,161	33,357	55,501,331	204,236,163
Cordage and twine	10.0	123	1,592	1,682,953	9,145,512
Cork products	3.9	35	166	199,347	389,254
Metal doors, frames, shutters	100	148	11,949	24,264,678	72,160,388
Elec. mach., appar. and supply	7.0	1,802	28,354	43,905,747	160,402,571
Engines, turbines, tractors	3.3	199	2,383	4,189,725	15,275,783
Explosives	13.0	95	836	1,392,856	9,430,164
Felt goods; wool, hair, jute	4.1	54	254	411,537	1,846,847
Foundry & machine shop prods	5.6	8,605	30,541	52,940,477	157,668,416
Gas and electric fixtures	35.4	614	10,128	16,630,706	51,166,012
Gas and other meters	60.5	72	4,522	7,735,676	24,589,197
Glass	16.1	263	11,822	16,644,444	48,800,326
Hardware, n. e. c	29.3	485	17,653	25,628,950	67,160,113
Steel works & rolling mills	18.3	486	78,970	145,018,032	616,699,637
Lime	42.7	237	4,031	4,921,442	15,293,286
Linoleum	100.0	7	6,894	11,030,773	57,250,449
Lumber and timber products	70.0	12,915	317,000	334,616,267	891,430,624
Marble, granite, slate and other stone products	62.3	1,881	28,196	50,797,372	119,638,585
Nails, spikes, (non-roll. mill)	60	55	1,448	2,159,059	7,712,847
Non-ferrous metal products	2.8	1,225	2,576	4,240,799	25,696,163
Paints and varnishes	55.0	1,063	23,283	44,153,109	312,936,711
Paper	4.1	685	4,719	7,298,913	39,242,908
Prepared paving materials	100.0	126	2,876	5,067,631	29,275,369
Planing mill products	70.0	4,849	76,905	113,717,211	387,508,449
Plumbers' supplies (exc. pipe & vitreous china ware)	100.0	255	31,353	46,567,677	129,673,784
Roofing materials	100.0	102	7,426	12,456,085	103,506,090
Rubber goods	2.2	412	1,024	1,469,346	5,294,105
Sand-lime brick	100.0	40	707	1,146,853	2,970,923
Saws	34.9	81	1,786	2,792,603	7,886,947
Screw machine products	3.6	273	808	1,405,870	3,779,466
Steam & other packing, pipe and boiler covering	6.6	171	575	910,335	3,007,394
Steam fittings & heating app	100	240	45,945	75,502,208	228,310,254
Stoves & warm air furnaces	20.4	609	11,182	17,766,897	55,358,466
Struc. & ornam. iron & steel (not made in rolling mills)	100.0	1,482	69,332	126,867,125	477,036,827
Tools, n.e.c.	15.7	534	3,627	5,581,767	15,568,992
Wall paper	100.0	56	5,421	8,282,817	30,007,649
Wall plaster, wall board, and floor composition	100.0	236	8,994	13,820,652	70,662,746
Window & door screens & weatherstrip	100.0	280	5,055	7,373,979	24,451,700
Wire	11.5	81	2,877	4,840,282	24,680,329
Wirework, n.e.c.	19.9	547	5,155	7,462,682	29,261,922
Wood preserving	100.0	199	14,497	18,561,537	190,944,645
Wrought pipe, not made in rolling mills	40.0	50	5,136	9,262,922	49,863,586
Total — Construction Materials Equipment and Supplies	.....	48,927	1,097,117	$1,580,484,876	$5,532,590,212
All manufacturing industries	.....	210,959	10,330,728	15,216,037,315	70,434,863,440
Construction manufacture per cent of all manufacture	....	23.1	10.6	10.3	7.9

Source: Compiled from Census of Manufactures, Bureau of the Census.

TABLE III— RAW MATERIALS AND SEMI-MANUFACTURES PRODUCED FOR THE MANUFACTURE OF CONSTRUCTION MATERIALS, EQUIPMENT AND SUPPLIES, 1929

Industry	Per Cent Cons.	No. Estab.	No. Employees	Payroll	Value Products*
Raw Materials:					
Copper mining	14.3	180	6,870	$11,917,068	$40,542,984
Iron ore	19.9	208	6,169	9,404,074	39,269,575
Stone	74.1	2,458	39,352	50,800,271	127,161,116
Sand and gravel	98.0	1,165	19,517	32,855,703	100,265,676
Clay	60.6	236	2,772	2,787,002	6,516,588
Gypsum	96.4	63	2,134	2,828,754	5,533,541
Asphalt and bituminous rock	95.0	25	1,242	1,748,348	4,867,644
Lead mining	13.0	171	1,951	3,310,266	8,783,031
Semi-manufactures:					
Blast furnaces	19.2	105	5,256	9,370,548	148,173,050
Copper smelting and refining	14.3	26	2,365	4,221,317	142,254,092
Lead smelting and refining	13.0	19	740	1,271,861	30,174,951
Petroleum refining (asphalt)	1.7	390	1,605	2,800,833	44,339,640
Totals	....	5,046	89,973	$133,316,075	$422,527,097*

Rated horsepower for these industries was 1,727,009.

*The "Value of Products" figure given is a net, non-pyramiding total; it was obtained by adding together the "Value of Products of the Raw Materials" group, the "Value Added by Manufacture" of blast furnaces, copper smelting and refining, lead smelting and refining, and the "Value of Products" for petroleum refining; the gross value of products for the whole group was $697,881,888.

Sources: The "Raw Materials" group was compiled from the Census of Mines and Quarries, Bureau of the Census; the "Semi-manufactures" group is from the Census of Manufactures, Bureau of the Census.

N.B. The production of lumber and timber in logging camps is properly attributable to the raw materials division; this operation, however, is reported in the Census of Manufactures together with planing and finishing operations. It is included, consequently, in the second division of this survey, "Manufacture of materials, equipment and supplies for construction."

TABLE IV — TRANSPORTATION OF RAW AND FINISHED CONSTRUCTION MATERIALS BY CLASS I STEAM RAILROADS IN 1929

Commodity	—Revenue Freight Originated— Carloads	Tons	Revenue
GROUP III			
Anthracite coal	18,639	935,108	$2,117,990
Bituminous coal	317,480	16,943,574	38,078,753
Iron ore	296,064	16,342,394	19,167,183
Copper ore and concentrates	18,146	1,009,102	429,704
Sand & gravel (other than glass)	1,511,267	81,408,069	69,420,839
Stone, broken, ground, crushed	622,472	33,529,078	29,860,479
Stone, rough, N.O.S.	79,348	3,654,466	6,080,704
Stone, finished, N.O.S.	14,565	476,295	2,525,393
Asphalt, natural, by-product, petrol	88,596	3,036,572	13,026,464
Lead ore and concentrates	3,327	175,864	242,899
Products of mines, N.O.S.	78,061	3,960,248	6,275,385
GROUP IV			
Logs	660,080	21,405,743	14,972,082
Posts, poles and piling	152,742	4,275,946	19,969,440
Ties, railroad	130,866	4,024,144	12,450,385
Lumber, shingles and lath	978,215	26,188,996	171,343,736
GROUP V			
Fuel, road, residual oils, N.O.S.	19,985	673,876	2,614,856
Iron, pig	29,872	1,592,081	3,039,042
Rails, fastenings, frogs, switches	63,234	2,658,312	8,699,149
Cast iron pipe and fittings	59,223	1,410,832	9,243,142
Iron and steel pipe and fittings, N.O.S.	76,954	2,362,432	20,563,880
Iron and steel; nails and wire, not woven	46,124	1,124,049	6,114,575
Iron and steel; Class 5, N.O.S.	208,371	6,849,181	28,952,020
Copper: ingot, matte, and pig	3,461	158,460	1,225,041
Machinery and boilers	12,345	220,133	2,208,511
Cement, natural or Portland	659,797	25,006,203	67,083,562
Brick, common	164,774	6,133,482	11,928,377
Brick, N.O.S., and building tile	172,569	6,271,385	16,974,469
Artificial stone, N.O.S.	24,443	733,226	2,284,365
Lime, common (quick or slaked)	50,121	1,143,889	3,619,808
Plaster, and dry kalsomine	79,644	2,124,257	8,743,567
Sewer pipe and drain tile, not metal	146,321	2,708,023	9,742,533
Tractors and parts	3,809	54,836	691,854
Explosives, N.O.S.	2,584	41,759	763,094
Paint in oil, and varnishes	13,537	291,271	2,391,034
Lead and zinc; ingot, pig, bar	3,928	158,305	1,194,933
Paperboard, pulpboard, wallboard	11,460	251,347	1,312,659
Building paper and roofing materials	105,224	2,324,998	12,183,396
Building woodwork (millwork)	25,326	452,106	4,715,515
Glass, flat, other than plate	12,523	352,443	2,601,344
Manufactures and miscell., N.O.S.	201,798	4,243,066	26,118,472
Total Construction Freight	7,167,331	286,701,871	$660,970,634
All freight traffic 1929	36,821,868	1,303,047,736	4,451,890,050
Construction freight, per cent of total	19.5	22.0	14.8

N.B. Commodity figures are not necessarily total for the particular materials; where commodity has other uses in addition to construction, the percentage for construction, determined in the manufacturing and raw materials divisions of this survey, has been applied.

Source: Computed from Interstate Commerce Commission Freight Report.

TABLE V—EMPLOYMENT FURNISHED BY THE TRANSPORTATION OF CONSTRUCTION MATERIALS, EQUIPMENT AND SUPPLIES, RAILROADS, 1929

Class	No. Emp.	Payroll
Group I —Executive officers and staff assistants	2,265	$13,930,837
Group II —Professional, clerical and general	39,002	67,437,897
Group III—Maintenance	6,344	11,125,835
Group IV—Maintenance of equipment and stores	64,369	108,254,251
Group V..—Transportation other than train, engine, yard	24,740	37,902,048
Group VI —(a) Transportation (yardmasters, etc.)	3,160	7,524,266
(b) Transportation (train and engine)	43,407	107,513,749
Total—Attributable to Transportation of Construction Goods	183,287	$353,688,883
All railroad employees	1,686,769	$2,940,868,690
Construction per cent of total	10.9	12.0

Source: Interstate Commerce Commission. These figures have been determined by studying separately approximately 100 classes of railroad employees, and apportioning a percentage (freight revenue is 78.4% of all railroad revenue) of each class to freight transportation. The percentage by carloads of construction freight of the total freight is 19.5 (see table); accordingly this percentage was then applied, wherever necessary, to number of employees and payroll for each class attributable to freight transportation to determine figures for *Construction freight* transportation.

and the manufacture of materials engaged a quarter of the total number, or 1,100,000 persons. The activities of these construction dependent employees were directed to a program of $11,000,000,000, and of this amount $7,000,000,000 went for payrolls in the different divisions of the industry.

Raw-materials production

The raw materials group is the first logical division of the survey, for these interests take the first step in the long construction process. The division covers production of stone, sand and gravel, clay, asphalt, and the metallic ores, and also certain semi-manufactures—products which have had some treatment, but which are not yet in finished form for consumption on the construction site.

These raw materials establishments are widely dispersed, ranging from the iron ore mines of the Lake Superior district to the copper mines of Arizona, and from gravel pits in country towns to Pittsburgh blast furnaces. Employment for this crude materials production is small compared to the 4,500,000 figure for the whole construction industry, but it must be remembered that the raw materials operations are carried on mainly in small towns which depend on this mining and quarrying for their existence.

In general, in this division and in the one following—manufacture of materials, equipment and supplies—percentage for construction has been determined by the only possible common denominator—value of products. The value of each material going to construction has been computed, and the resulting percentage of the total value of products of the industry applied to other data, such as number of employees, payroll and horsepower.

Employment in mines and quarries, then, for the production of crude construction materials was 90,000, with a payroll of $133,000,000. (Table III). The gross value of products was $700,000,000, which, however, scales down to a net value of $422,000,000.

Logging camp operations rightfully belong in this division, but the Census of Manufactures report them together with sawmill and finishing operations and the segregation is difficult. Consequently the figures for lumber camps are included in the lumber and timber products industry of the manufacturing division of this survey.

Manufacture of materials, equipment and supplies

Fifty industries, covering the manufacture of every type of material used in construction, have been analyzed for this section of the survey. Table II lists these industries and presents figures for each of them. A tenth of the manufacturing workers in the country, 1,100,000 persons, were engaged in the

production of these materials, and the process was carried on in almost a quarter of the factories and mills of the country. Eight per cent by value of all manufactured products in 1929 were construction materials, the gross value of these products being $5,500,000,000. This figure, however, contains a great amount of duplication. For example, the value of cement used in the concrete products industry is reported twice—once by the cement manufacturer in his report, and later reflected in the value of products as reported by concrete products manufacturers. For this reason "Value Added by Manufacturer" (value of products less cost of materials and fuel) is perhaps a better standard of measurement, for it is non-duplicating. This "value added" for construction materials was $3,047,000,000. The rated horsepower of machinery used in manufacturing these materials was 6,500,000.

The accompanying map illustrates the penetration of these materials industries into every state of the nation. The density indications may, on first examination, be questioned, for they are at variance with accepted ideas of industrial distribution. Many states considered negligible in the general industrial picture show up heavily as producers of construction materials. Thus Mississippi, Louisiana, Oregon and Washington are high above Massachusetts and Missouri. The paradox is attributable to lumber production, in which the South and Northwest predominate. Alabama combines steel with her lumber production to rank high as a construction supplier.

The list is headed by Pennsylvania, Ohio, New York, and Illinois, each with over 50,000 employed in these industries. Pennsylvania alone employed 133,000 persons.

The production of coal for the manufacture of construction materials engaged 24,000 miners, and 5 per cent of all coal mined was burned in this production.

The general statement on the raw materials group holds true for the manufacturing division. Value of products has been the basis for determining the construction percentage of any given industry, and number of employees, payrolls and the like computed from this percentage.

TABLE VI—WHOLESALE AND RETAIL DISTRIBUTION OF CONSTRUCTION MATERIALS, EQUIPMENT AND SUPPLIES, 1929

Commodity	Per Cent for Constr.	No. Establ.	No. Employees	Payroll	Net Sales
Wholesale:					
Explosives	13.0	118	184	$471,332	$13,942,733
Paints and varnishes	55.0	1,027	6,914	15,455,771	167,749,508
Logs piles and posts	70.0	351	1,371	1,719,527	33,666,529
Railroad ties	100.0	157	572	600,737	9,749,039
Hardware	29.3	1,789	13,224	24,781,441	253,784,391
Constr. and building materials (exc. wood and met.)	100.0	3,222	46,979	97,127,380	1,009,946,114
Lumber and millwork	60.0	2,291	14,924	30,975,156	680,523,678
Constr. equip. and suppl.	100.0	498	4,059	9,927,481	132,689,655
Plumbers' equip. and suppl.	100.0	22	315	629,060	4,087,561
Wall paper	100.0	232	2,612	4,550,500	32,188,220
Plumb. and heating equip. and suppl. (excl. stoves)	100.0	2,649	34,876	73,653,649	785,818,087
Cordage and twine	10.0	105	122	288,023	5,428,112
Electrical (all types)	7.0	3,870	4,895	9,772,925	170,459,697
Coal	4.2	1,343	593	1,393,129	48,732,194
Iron and steel (exc. scrap)	18.3	1,643	3,624	9,895,531	519,713,500
Sheet metal products	90.0	253	2,442	5,299,075	182,213,167
Wholesale Totals—Construction	.....	19,570	137,706	$286,539,717	$4,050,592,185
All wholesaling	.....	169,702	1,605,042	3,010,129,535	69,291,547,604
Per Cent const. of total	.....	.....	8.6	.....	5.8
Retail:					
Lumber and building material dealers	100.0	16,911	117,003	166,471,494	1,471,744,992
Lumber and hardware deal.	100.0	6,139	35,615	47,703,708	457,659,775
Roofing	100.0	2,868	12,507	11,977,965	46,160,499
Other building mater.	100.0	459	1,690	1,301,073	5,718,391
Elec. shops (without radio)	100.0	4,858	20,938	24,095,731	110,131,328
Heating appliance and oil burners	100.0	1,915	11,920	20,509,706	70,736,608
Plumbing shops—heating and ventilating	100.0	10,794	53,969	61,955,407	263,540,222
Paint and glass	100.0	8,282	30,414	32,257,796	185,819,822
Hardware stores	29.3	25,330	24,224	21,922,040	206,873,479
Build. mater. and hardware—2nd hand	100.0	429	2,338	2,191,604	9,286,140
Retail Totals—Construction	.....	77,985	309,618	$390,386,524	$2,827,671,256
All retailing	.....	1,543,158	6,020,747	$5,351,057,475	$49,114,653,269
Per cent const. of total	.....	.....	5.1	.....	5.8

Source: Census of Wholesalers and Retailers, Census Bureau.

TABLE VII — GENERAL SUMMARY OF ALL CONSTRUCTION IN THE UNITED STATES, 1929

Type	Value	Per Cent of Total
Residential	$2,127,000,000	20.2
Commercial	1,031,000,000	9.8
Factories	606,000,000	5.8
Theaters, clubs, lodges, relig. and memorial	224,000,000	2.1
Farm construction	463,000,000	4.4
Total private	4,451,000,000	42.4
Railroads	1,370,000,000	13.2
Electric power companies	906,000,000	8.6
Telephone companies	795,000,000	7.6
Electric R.R. companies	194,000,000	1.8
Group total	3,265,000,000	31.1
Cities	1,339,000,000	12.8
Counties	556,000,000	5.3
States (excluding federal aid)	576,000,000	5.5
Federal (including federal aid, excl. D. C.)	305,000,000	2.9
Total public	2,776,000,000	26.5
GRAND TOTAL	$10,492,000,000	....

This table was compiled by the Federal Employment Stabilization Board from reports to the F. W. Dodge Corporation, to the Department of Agriculture, the Bureau of the Census, and the Federal Employment Stabilization Board.

The table is incomplete for no reports were available on pipe line company, gas company, telegraph company or waterworks company construction. Studies being made by Dr. A. D. Morehouse point to a construction total in 1929 of close to 11½ billions. Throughout the Construction League survey the figure of $11,000,000,000 is used.

The Census Bureau's Construction Census in 1929 reports a net construction figure of contractors of $5,759,774,790, or 52.5% of all const.

NUMBER OF EMPLOYEES — ALL CONSTRUCTION

Contract: Number employed, 1,252,466. Payroll, $2,233,646,822.
(Computed from Census of Construction, Bureau of the Census.)

Railroad: Number employed, 367,472. Payroll, $391,316,419.
(Computed from Interstate Commerce Commission report.)

Other construction (this includes all construction except contract and railroad. As no reports are available on this work, an estimate of number of employees must be made.)
Census of Contract Construction shows an average of $5,730 of work per wage earner; an average of $26,662 of work per salaried worker; and an average of combined wages and salaries of $1,813. Applying these figures to the $3,870,225,210 of construction done directly, the results are:

Wage earners	675,432
Salaried workers	145,159
Total	820,591
Total payroll	$1,487,731,483

* * * *

GRAND TOTAL — All employees	2,440,529
Total payroll	$4,112,694,724

Transportation of materials

That the railroads lean heavily on construction is evidenced by the fact that one of every five carloads of freight moved in 1929 carried construction material (Table IV). Such materials constituted 22 per cent of total freight tonnage and provided 15 per cent of all freight revenue. Number of construction freight carloads was 7,100,000. Eleven per cent of all railroad employees were occupied with this construction goods transportation, 180,000 persons, and their payroll was 12 per cent of the total railroad payroll. (Table V).

The percentages determined in the

TABLE VIII—DESIGN OF STRUCTURES AND PROJECTS, UNITED STATES SUMMARY, 1929

	Per Cent	Number
Architects	100.0	22,000
Designers and draftsmen	33.6	33,742
(Total excludes designers and draftsmen in industrial and trade establishments)		
Civil engineers and surveyors.	87.1	88,919
(Total excludes civil engineers and surveyors engaged by industrial establishments)		
Grand Total—Division V	.....	144,661

Source: Census of Occupation, Bureau of the Census. Figures as of April 1, 1930.

TABLE IX — SUMMARY OF CONTRACT CONSTRUCTION, 1929

Total number of establishments reporting		144,396	
Doing at least $25,000 of business		30,597	21.2%
Doing less than $25,000 of business		113,799	78.8%
Value of construction business		$7,285,720,244	
Est. doing at least $25,000 of business		6,250,266,665	85.8%
Est. doing less than $25,000 of business		1,035,453,579	14.2%
Net value of construction work (value of work done by own force)			
Total		$5,778,453,565	83.0%
Est. doing at least $25,000 of business		4,794,722,665	83.0%
Est. doing less than $25,000 of business		983,680,900	17.0%
Value of business done by:			
Operative builders		153,524,000	2.1%
General contractors		4,569,070,770	62.7%
Subcontractors		2,560,425,395	35.2%
Employees:	**Number**	**Payroll**	**Per Cent by Emp.**
Total — all classes	1,252,466	$2,233,646,822	....
Operative builders	9,422	17,550,330	0.8
General contractors	732,987	1,255,536,579	58.5
Subcontractors	510,057	960,559,913	40.7
Wage earners — total	1,008,775	1,781,498,962	....
Operative builders	7,369	13,146,000	....
General Contractors	625,956	1,021,029,946	....
Subcontractors	375,450	747,323,016	....
Average wage	$1,766		
Oper. build.	1,784		
Gen. contr.	1,631		
Subcontractors	1,900		
	Average amount of work per wage-earner		$5,730
Salaried workers — total	216,635	415,886,534	
Operative builders	1,440	3,957,940	Average Salary, $2,922*
General contractors	93,211	218,898,370	
Subcontractors	121,984	193,010,224	
Proprietors and firm members* — total	27,056	36,281,776	
Operative builders	613	446,840	....
General contractors	13,820	15,608,263	....
Subcontractors	12,623	20,226,673	....

*For establishments doing at least $25,000 of business in 1929.

Number of employees by classes of establishments:	No. Emp.	Per Cent of Total	Payroll	Per Cent of Total
Est. doing at least $25,000	958,543	76.6	$1,805,890,860	80.8
Est. doing less than $25,000	293,923	23.4	427,755,962	19.2

Value of business by types of contractors (estab doing at least $25,000)
(Values expressed in thousands)

Type	Value	Per Cent of Total
Operative builders	153,524	2.5
Gen. contractors	4,217,367	67.5
Subcontractors	1,879,376	30.0

Types of General Contractors	No. Estab.	Value	Per Cent of General Contractors', Total
Total	14,766	4,217,367	100.0
Building	10,131	2,622,047	62.2
Build. not spec.	7,062	2,149,941	....
Comm. build. only	539	132,627	....
Manuf. build. only	75	93,828	....
Resid. build. only	2,455	245,651	....
Highway	1,514	443,064	10.5
Bridge and culvert	475	137,206	3.3
Grading	396	47,733	1.1
Street paving	984	282,760	6.7
Sewer, gas, conduit	494	115,153	2.7
Dam and reservoir	87	28,483	0.7
Waterworks	105	44,210	1.0
Dredging	193	104,463	2.5
Levee	34	5,631	0.1
Railroad	137	105,302	2.5
Foundation	56	8,282	0.2
Central station, light and power pl	86	158,160	3.8
Air transport	7	2,416	0.1
Refuse disposal	7	1,721	....
Oil and gas pipe	22	29,507	0.7
Subway	24	71,818	1.7
Vehic. subway and tunn	3	3,772	0.1
Telephone, radio and miscellaneous	11	5,639	0.1

Types of Subcontracts	No. Estab.	Value	Per Cent of Sub., Total
Total	15,081	1,879,376	100.0
Carpent. and floor	382	36,611	1.9
Concreting	477	66,312	3.5
Electrical	1,928	198,487	10.6
Elevator	129	77,376	4.1
Heat. and plumb	5,583	611,886	32.7
Masonry	673	85,023	4.5
Paint. and decor.	1,102	83,899	4.5
Glass and glazing	154	16,653	0.9
Plaster. and lath.	623	83,313	4.4
Roof. and sheet-metal	1,697	145,095	7.7
Steel erection	300	135,716	7.2
Stonework	173	27,085	1.4
Marble and tiling	851	112,950	6.0
Wrecking	44	3,816	0.2
Excavating	293	39,813	2.1
Ornamental iron	227	36,077	1.9
Highway	26	4,174	0.2
Bridge and culvert	14	1,496	0.1
Grading	59	5,636	0.3
Street paving	18	3,324	0.2
Sewer, gas, conduit	14	5,480	0.3
Dam, waterworks	4	334	....
Dredging	13	2,346	0.1
Soundproofing	26	5,556	0.3
Railroad	3	934	0.1
Foundation	25	22,950	1.2
Power plant	4	1,064	0.1
Metal work, pipe line	138	52,272	2.8
Rental of equip., etc.	17	2,508	0.1
Flooring, n.e.s.	81	10,760	0.6
Miscellaneous	3	429	....

raw materials and manufacturing divisions of the study have been applied to the different commodities in this division to determine the carloads, tonnage and revenue of construction freight.

Distribution of materials

Nine per cent of the country's wholesalers were engaged in the distribution of $4,000,000,000 of construction materials. The number of persons was 138,000 and the materials they distributed were 6 per cent of the value of all goods sold at wholesale. Five per cent of all retailers were employed in selling construction materials, 310,000 people. The value of the goods sold was $2,800,000,000, 6 per cent of all retail sales.

Design

A total of 144,000 professional men derived their livelihood from the designing and supervising of construction projects. The number of architects was 22,000, engineers 89,000 and designers and draftsmen 33,000 (Table VIII). These figures represent 87 per cent of the engineers of the country and 33 per cent of the designers and draftsmen.

Construction

Estimates of the Federal Employment Stabilization Board place the value of construction in 1929 at 10½ billions of dollars, but studies being made by A. D. Morehouse, who directed the work of the Census of Construction, will swell the figure to approximately 11½ billions. Throughout this survey the figure of $11,000,000,000 has been used. Private construction was 42 per cent of the total, and government work, cities, counties, states and federal, accounted for 26 per cent (Table VII). The Census of Construction reports the value of contract work as $5,700,000,000, or 52 per cent of the total, the remainder having been done directly by governmental agencies, railroads, public utilities companies, farms, and the like.

Total employment on the construction site was 2,400,000, and the payroll $4,000,000,000. Contractors employed a million and a quarter of all persons engaged in construction work, paying an average wage of $1,766. Each wage-earner accounted for $5,730 of work (Table IX). General contractors did 67 per cent of the contract construction business, and of their $4,200,000,000, 62 per cent was building, and 10 per cent highway construction. The subcontractors accounted for 30 per cent of the gross contract construction business, with heating and plumbing the most important item of subcontract work (33 per cent).

The accompanying map indicates the diffusion of contract construction employment. New York led in number of employees with 197,000, followed by Pennsylvania, Illinois, California and Ohio.

New Cutoffs Being Made on the Mississippi

Dike break at Leland Neck in the Greenville Bends hastens channel-straightening program for the Mississippi River from the Arkansas River to Vicksburg

ON JUNE 3 a 12-ft. overbank high water broke through the permeable pile dike built after the flood of 1929 to prevent a cutoff at Leland Neck on the longest loop of the famous Greenville Bends of the Mississippi River. The dike break occurred fortunately at the crest of the high water, and the receding stages did not further develop a break-through channel. It was evident, however, that future flood stages would form a cutoff unless large expenditures were made for preventive works. Moreover, this cutoff would be uncontrolled. It seemed a sound measure, therefore, particularly as such a cutoff was included in the future river-straightening plans to control the cut-through in time and location. This was done by removing more of the dike and making pilot cuts across the neck. Details of this operation are given in a succeeding section.

The Greenville Bends

To comprehend the full significance of the new cutoff in the history of the control of the Mississippi River, an account of the stretch of the river known as the Greenville Bends is necessary. This river stretch is shown by Fig. 1. The "bends" proper embrace about 40 river miles and include four river loops separated at their bases by very narrow necks of land. Throughout these 40 miles of river the average mean-low water slope is 0.205 ft. per mile, whereas the average slope from Cairo to Red River landing is 0.35 ft. per mile.

Under such conditions rivers tend to restore the flatter slopes of meanderings to the river average. The records show that this tendency has been operative at the Greenville Bends. Half a dozen successive surveys since 1765 show a constant downstream migration of the bends and an accentuation of their curvature. All the necks have lengthened and narrowed. It is probable that cutoffs would have occurred long ago had they not been prevented by the river engineers. The protective dike and revetment operations are indicated on Fig. 1. In spite of these, the efforts of the river to break through are clearly indicated by the series of water-filled depressions or "blue holes" scoured in the necks by successive floods.

This scour development has been particularly notable at Leland Neck. Here a series of existing or vestigial blue holes extends from the base levee along the neck for about 2½ miles. The largest of these was formed by the flood of 1929. A total of 2,500,000 cu.yd. was then scoured out, leaving a hole 80 ft. deep, 2,600 ft. long and 600 ft. wide. The progressive advance of the scour from the base levee toward the end of the loop (Chicot Point) has been due to attempts of the river to delve out a passage around the ends of successive extensions of the cutoff dike projecting from the base levee, as shown by Fig. 1.

Despite the dike works, which will be considered later, and the revetment of scouring banks, the determined policy, prior to the last three years, of preventing cutoffs at all hazards has become progressively more difficult to maintain at the Greenville Bends. The necks have become so narrow and the front exposed so extended, that defense calls for tremendously costly operations. Under the present régime of river control it is being calculatedly abandoned, but with the purpose of controlling positively the loop-elimination process.

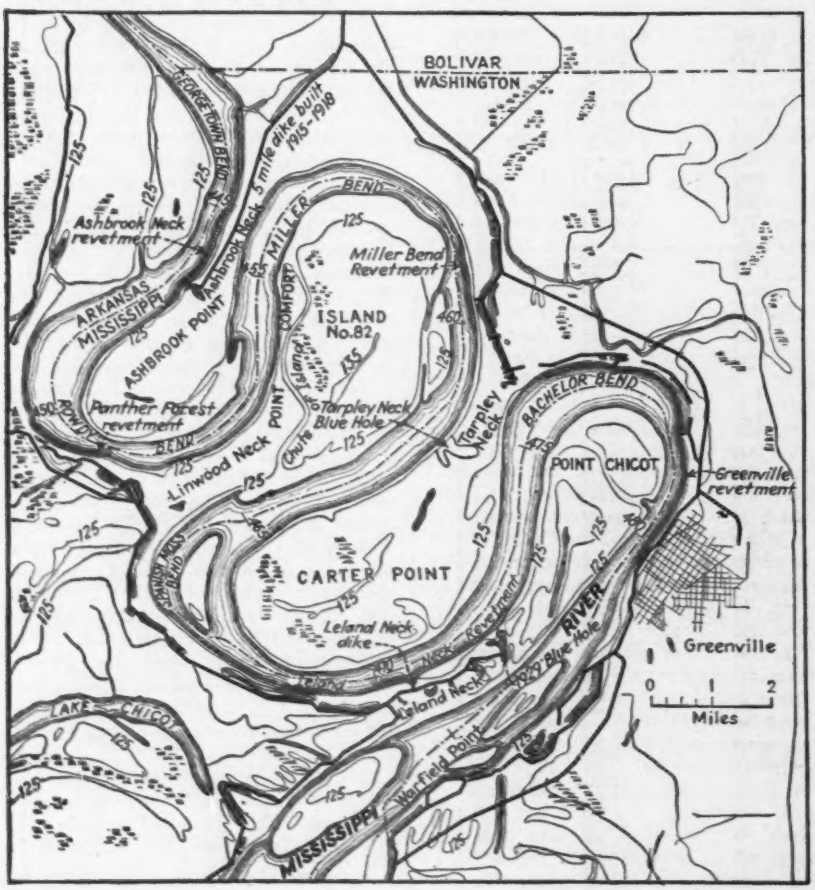


Fig. 1—The four river loops of the famous Greenville Bends of the Mississippi River, maintained inviolate for half a century by revetment and dike construction, have been broken by a cutoff at Leland Neck and another planned at Tarpley Neck.

Cutoff dikes

In the former policy of preventing cutoffs, dikes extending from base levees out toward the points of loops were accepted practice. The object was to dam the flow across the necks of loops and divert it around the loop. Generally these dikes were built to less height and cross-section than the base levees, but, like them, they were earth embankments. Two of these dikes were constructed at Ashbrook Neck and at Leland Neck in the Greenville Bends. Both of these dikes are shown by Fig. 1.

The Leland Neck dike is of especial interest because of its importance, the difficulty of its maintenance and the fact of the recent break. In 1882 this neck was 5,500 ft. wide at its narrowest part, but by 1903 this width had been reduced to 3,500 ft. Floodflow in that year caused deep scour across the neck near the controlling levee line at its base. Immediately after the flood an earth dike 6,250 ft. long was built out along the neck from the base levee. The

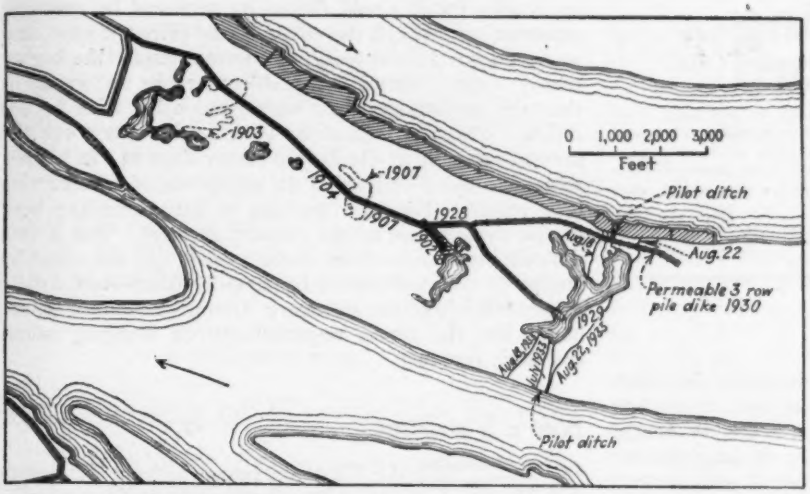


Fig. 2—Section of Leland Neck, showing successive blue holes, scoured as dike was extended, and location of final dike break and new cutoff channel.

high waters of 1904 to 1907 caused scour along the upper side of this dike and at its end, and after the flood of 1907 it was extended an additional 3,000 ft. The 1922 flood again caused deep scour around and beyond the end of the dike. In 1926 the riprap protection at the end of the dike was renewed, and a small brush and stone sill was built to prevent the extension of the blue hole at its end toward the upper bank of the bend. The flood of 1927 dangerously enlarged this blue hole, however, and the dike was thereafter extended an additional 4,000 ft., making its total length about 13,250 ft. The scour caused by the 1927 flood amounted to about 500,000 cu.yd., and the flood of 1929 threatened to cut off the neck.

The blue hole left by this last great flood has already been described. As indicated by Fig. 2, it had an arm 300 ft. wide which extended to within about 500 ft. from the upstream side of the neck. Some 330 ft. of the dike were destroyed during the flood of 1929, and an additional 1,200 ft. was badly wave-washed. It was evident that another high water would complete the cutoff unless effective steps were taken to prevent it. In 1929-'30 a permeable dike was built. This permeable dike left the old earth dike at a point 8,750 ft. from the base levee and extended along the bank on the upper side of the neck for a distance of 5,000 ft. For 4,600 ft. of its length, the top of the dike was 20 ft. above the ground surface. At its outer end, however, the grade of the top was gradually decreased over a distance of 400 ft. to a minimum of 10 ft. above the ground surface.

Structurally the dike consisted of a line of pile bents, 20 ft. wide and spaced 10 ft. apart. Each bent consisted of three piles driven 10 ft. on centers, in line normal to the axis of the dike. The piles were driven to a penetration of 20 ft., and the bents were substantially cross-braced. The upstream face of the dike consisted of horizontal 4x8-in. walings spaced 8 in. apart, and the downstream face consisted of vertical 4x4-in. screen poles spaced 8 in. on centers and bolted to horizontal stringers. The right-of-way 80 ft. wide was heavily paved with a reinforced-concrete mat to prevent scour through the structure. To check any flow of water along the dike, five short wing dikes were extended from its downstream face across the right-of-way and well into the woods below. These wings were timber cribs filled with concrete blocks. The upper lip of the 1929 blue hole was heavily riprapped to prevent its extension upstream into the right-of-way of the dike.

Developing the cutoffs

It was not desired at the present time to make a cutoff at Leland Neck, although it was included in the present plans for straightening the river. This cutoff and one at Tarpley Neck, where a break-through was recognized as inevitable in a few years, were planned to be held off until the river below could be prepared better to handle the changed flow. The dike break at Leland Neck forced the hands of the river engineers, and inauguration at once of cutoff work was decided on. Pilot channels were cut from the river into the blue hole both from above and below, as indicated on Fig. 2. These cuts were made in July, and in August they had widened as shown as of the dates given. The channel is steadily enlarging, and on Sept. 1 about one-third of the river flow was through the cutoff.

This cutoff projected other operations necessary to meet the changed flow. Revetment will be placed on Warfield Point below the new channel to prevent any extensive bank erosion. To prepare the river below Greenville to Vicksburg for the Leland cutoff and another that will occur in a few years at Tarpley Neck, cutoffs (Fig. 3) have been authorized at Worthington Point, Willow Point and Marshall Point. The excavation of pilot ditches at all three locations will be begun at an early date. All of these cutoffs are channel rectifications rather than true loop eliminations such as would be made by the river itself. It is expected to have these cuts open and water through them by the next high-water season.

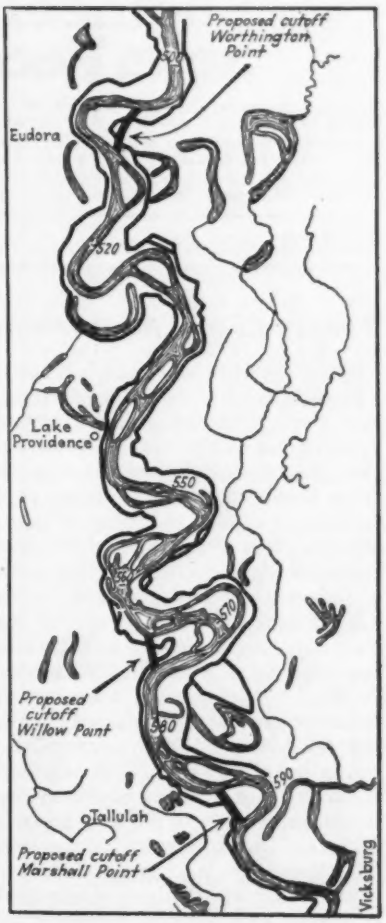


Fig. 3—River rectification under way below Greenville Bends to prepare the river for changed flow due to cutoffs at Leland and Tarpley Necks.

New Gas-Pipe Line to Run From Texas to Mexico City

One of the features of the six-year industrial-recovery plan for Mexico, which will be placed in motion soon with former president Plutarco Elias Calles as director, is the proposed construction of a natural-gas pipe line from the natural-gas fields of the lower border region of Texas, in Zapata and Starr counties, to Mexico City and intermediate towns. The new pipe line will parallel the existing one that runs from these fields to Monterey. The United Gas Co. and the Electric Bond and Share Co. will be parties to financing the project.

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Time to Change the Emphasis

TOO MUCH emphasis has been placed on the NRA objectives. Too little pressure has been applied to the PWA program. These statements are of course relative, but so far, in the administration of the Recovery Act, the tail has wagged the dog. Most of our energies have been directed to industry regimentation; too few have been applied to giving industry jobs. It is important that this situation be widely appreciated, for already there are those who are saying that the Act has proved a failure. Financial writers and extreme inflationists are particularly articulate in this respect. "The NRA," they say, "despite nationwide backing has not solved the unemployment problem. The next step, therefore, is inflation." Whether we need or will get more or less inflation can hardly be based on the false premise that the NRA has failed to accomplish something that was never its objective. NRA administers Title I of the Recovery Act. So far as the unemployment emergency is concerned this is the less important part of the Act. Title II, Public Works, is the unemployment reliever. The Recovery Act has not failed. It has not yet been given a chance. Without public works and a resumption of capital expenditures generally, the elaborate machinery that NRA has set up is an empty shell. It is time to change the emphasis of our endeavors.

Labor Questions of New Kind

IN THESE decisive days of industry planning and regulatory codes for recovery, the construction industries find themselves facing a special responsibility. Construction, one of the country's largest employers, suffered far more severely than most industries in the depression and made a heavy contribution to unemployment. Its three to four million employees make the construction code many times as weighty as the much discussed code of the coal industry, with its half million workers. What the construction code can accomplish toward reviving employment will have much to do with making the present incipient recovery permanent or otherwise, for the capital-investment industries, headed by construction, are the indispensable key to increased buying power. In view of this tremendously important re-employment objective, the most difficult part of the code task is to solve the labor problem in such a way as to stimulate renewed construction while being fair to the worker. In this respect the problem differs fundamentally from that of the consumer industries, which deal with necessaries; construction is not a necessity in the same sense—it may be undertaken or postponed, and will not become active except by the stimulus of attractive price. Here, then, the simple device of raising wage rates, upon which all prior work of the NRA has relied, is no longer applicable, for it would defeat its own end by checking construction through the effect of the price increase, and so would curtail the worker's income despite the higher hourly wage. Hence the problem before the group is the very opposite of that heretofore dealt with by the NRA. The effort must be to fix wage rates for the present moment at the lowest rather than at the highest level feasible, in order that the maximum of construction may result. This will operate to the advantage both of the worker and of the country at large. But it is a difficult problem, and one to be worked out not alone by engineers and contractors but also by labor—and finally by the officials of the Recovery Administration, in whose hands lies the broad responsibility of bringing about economic recovery.

Not a Yardstick

ANNOUNCEMENT of the rates which the Tennessee Valley Authority will require municipalities owning their own electric distribution systems to charge for power bought from it is the week's most momentous item of economic news, because of the TVA's avowed purpose of setting up a yardstick for measuring the rates of other utilities. That such a yardstick is much to be desired can hardly be questioned. But unfortunately the figures which the Authority has given out do not constitute such a yardstick. They are incomplete, and so raise question both as to how the rates were arrived at and as to whether a municipality can afford to abide by them. Power is to be sold to municipalities owning their own distribution systems at an average price of 7 mills per kilowatt hour—which being not a low figure for a wholesale price bears out the Authority's statement that it has made a serious attempt to compute costs on a basis comparable to that of a privately owned public utility which must provide for interest, amortization and taxes. But over against this relatively high wholesale rate the TVA has set up unusually low domestic rates which it will require municipalities to charge its customers. These rates range from 3 cents for the first 50 kilowatt hours to 4 mills for amounts beyond the first 200 kilowatts. No details of how these rates were determined are given. They do not appear to make due allowance for distribution and bookkeeping costs to the municipality. Further, they appear to subsidize the large consumer at the expense of the small one. Until *all* the figures are available it will be impossible to judge how accurate and hence how useful a yardstick is to be provided by the TVA.

PWA Grants

FINANCING of municipal construction work under the Recovery Act took on a new aspect last week when the Public Works Board approved direct grants of 30 per cent of the cost of labor and material upon a number of projects where the applicant expected to obtain the remainder of the cost of the project as a loan from sources other than the PWA. While such a development in connection with the loan provisions of the Recovery Act apparently was not anticipated by those who drew up the act, it is a logical development and quite within the spirit of the legislation. There is no reason why a community which still is able to borrow money at favorable rates should not have the benefit of the 30 per cent grant in financing needed facilities in these difficult times just

as does the community that is in a less favorable financial position. Otherwise the act would put a premium on bad city financing. Still another aspect of the Public Works Board's action is its possibility for expanding the construction program of cities that are in a position to finance their new projects fully. Consider a street repaving program, for example: If the city's budget calls for an expenditure of $100,000 the program can be expanded by nearly $30,000. In view of these effects the result of the Public Works Board's action should be to increase greatly the amount of municipal construction that will be carried out as a direct result of the loan provisions of the Recovery Act.

The Passing of Greenville Bends

ACCELERATION of the channel-straightening program contemplated in the new plans for the Mississippi River has been enforced by the river itself. The high water of early June, which carried away the protecting dike on Leland Neck, already half severed by the great pit scoured by the flood of 1929, put the river engineers to the necessity of retreating from this line of defense, which they had strengthened repeatedly for thirty years. Their hands thus being forced by the certainty of imminent defeat, they decided to accept the fortunes of war but to control the hostile advance. The incipient rupture of the endangered neck was therefore made complete by a dredged channel.

The circumstances as related in this issue have compelled a new alignment of the plans contemplated by the present river administration. Preparation of a rectified channel below Leland Neck, which normally would have preceded a designed cutoff in order to provide a better getaway channel for the changed flow, now must follow instead of precede the major loop elimination; the necessary point eliminations are already under way. The change from logical order of the straightening operations is not likely to result in any hazard, however, for cutoff development even after the actual break-through is a rather deliberate process (although at Leland Neck the action has been faster than common) and gives time for throwing up downstream defenses in the form of realigned channels and suitable bank protection.

Aside from any problems that may arise on account of revision of construction programs the operations described in this issue have importance as a guaranty of confidence in the bold plan, inaugurated in defiance of all tradition, of revising the channel of the Mississippi to conform to the requirements of an efficient hydraulic conduit. The four cutoffs below Vicksburg are now increased, and Tarpley Neck of the Greenville Bends is likely to be breached by highwater soon and perhaps may be artificially severed even earlier. Thus the first phase of the new control plan is in its concluding stage.

To many old river residents the beginning of the end of the Greenville Bends will appear as the loss of a most picturesque feature of the river. To river engineers it means the conclusion of one of the most fascinating battles of a generation—to maintain the old tradition of the sanctity of channel courses as they had endured during recorded history. Fortunately the Bends, except for the city of Greenville, mean little for human habitation, and Greenville will merely be put on a spur of the river instead of being on the main channel. The disadvantage is theoretical rather than actual, as proved by the little economic effect on Vicksburg when it was similarly isolated by the Centennial Cutoff of 1876. The passing of the Greenville Bends, famous since navigation of the river by white men began, is unlikely to cause other than sentimental regrets.

Floods and the State

IN THE abnormal rains of recent weeks numerous streams throughout the country have given new evidence, by damage and destruction along their banks, of the great size of the flood-prevention and protection problems that are waiting to be dealt with. Some of these problems are obviously local and should be handled by the properties and communities in which the damage occurs. But this is not true of all. In many cases the region which is or should be concerned with flood-protective measures extends far from the zone of physical damage. For example, flash floods in the torrents and brooks at the headwaters of an important stream do extensive damage to roads, bridges and hillsides in these upper reaches, while farther downstream the same floods are hardly noticed. But the thinly settled countryside around the headwaters lacks the resources necessary for effective control of the stream, and in addition should not be expected to bear the burden since the stream is an interest of the whole country or more often of the state.

Until recently it was universal practice to charge the township with maintaining all but major roads and bridges, regardless of the severity of the burden thus imposed. That system has been changed for the better in many states by the transfer of all road and bridge maintenance to larger governmental units. Similar action should be taken everywhere, but it does not go far enough. The control and regulation of streamflow ought to be recognized as the state's responsibility in its entire extent.

A good beginning toward this objective can be made if the state will organize an adequate division for the study of stream and floodflow. The work of such an organization would soon furnish data permitting an adequate appraisal of the character and extent of the problem, and on the basis of this knowledge intelligent planning for active attack on flood hazards could be carried out. The first requirement for progress is comprehensive engineering study of the facts. This is plainly the duty of the state.

Such study by competent state engineering organizations is the more necessary because, as in the case of shore damage and shore protection, knowledge of proper procedure is still sadly incomplete. In shore protection this lack of knowledge relates primarily to the mechanics of protective construction. In stream control the technical methods are much more fully known, but on the other hand the economics of the subject are almost completely unexplored. What methods of control are preferable and justified in a given case, how the costs should be allocated among different interests with regard for justice and feasibility, and to what extent flood control should be geared in with water conservation—such questions permit of no broad or academic answers. It is only by thorough individual study that the right foundation can be laid for policy determination, from which effective control operations can be developed. The magnitude as well as the intrinsic importance of stream damage is full warrant for state action in this direction.

NEWS OF THE WEEK

Large Allotments of Federal Funds Made by PWA

ALLOTMENTS of funds totaling over $55,500,000 were made by the Public Works Board during the past week. The loans include the first loan to a private corporation under the Recovery Act, loans to municipalities and direct grants for federal work.

The largest single item is the allotment of $22,000,000 for the construction of locks and dams on the Upper Mississippi. This is in addition to the previous allotment of $11,500,000 for dredging in the 9-ft. channel between St. Louis and Minneapolis.

Among the allotments for other federal work were $1,125,285 to the War Department for the construction and repair of 16 army posts; $281,000 for dredging and retaining wall construction at Ocean City Harbor and Sinepuxant Bay, Md., which is to be supplemented by $500,000 from the state to repair storm damage; $68,700 for dredging in Knapps Narrows, Md.; $60,000 for flood control on the Rio Grande near San Benito, Tex.; $47,000 for a shelter harbor at Deals Island in Chesapeake Bay; $12,000 for repairs to a sea wall at Fort McHenry National Park, and $600,000 for the Mall Development Program at Washington, D. C. A sum of $1,500,000 also was allocated to Puerto Rico, $1,000,000 for highways and $500,000 for land reclamation.

The first loan to a private corporation for a public project made by the PWA was a loan of $250,000 to the Jacksonville Produce Terminal, Inc., for a shipside terminal market at Jacksonville, Fla.

Non-federal projects

Thirty non-federal projects received loans totaling $29,635,000. They are given in the accompanying table.

State loan applications

Applications for PWA loans totaling $1,518,000 for 14 projects in Oklahoma have been received by the state advisory board. The loans cover chiefly schools and other public buildings, but two power plants and distribution systems, one costing $300,000 for Cushing and one costing $150,000 for Hominy, are included.

Applications for PWA loans filed with the state engineer of the PWA in Texas now total in the neighborhood of $5,000,000. The larger items include buildings costing $2,800,000 for the University of Texas at Austin, a reservoir and irrigation system at Abilene costing $2,000,000 and $355,000 for improvements to the buildings of the College of Industrial Arts at Denton. Plans are being made to reorganize the LaSalle County Water Improvement District No. 1 in order to obtain federal funds for carrying on that work. A bond issue of $7,000,000 was authorized in 1925 but the bonds were never sold. About $100,000 has been spent on the construction of a dam ten miles west of Cotulla.

Loans totalling nearly $20,000,000 for building construction throughout the state of Michigan have been proposed by the Governor to the state advisory board of Michigan. The work covers state hospitals and training schools and National Guard armories.

Governor Lowden Not to Serve on Mississippi River Board

Frank O. Lowden, former Governor of Illinois, has declined the chairmanship of the committee recently named by Secretary Ickes (*ENR*, Sept. 14, p. 331) to make a study of the entire Mississippi River work to select projects for inclusion in the public works program.

DETAILS OF PWA LOANS AND GRANTS FOR NON-FEDERAL WORK

Recipient	Project	Loan-Grant	Amount
Sacramento, California	Water	L and G	$11,700,000
Fargo, North Dakota	Sewage	L and G	512,000
Fort Atkinson, Wisconsin	Sewage	G	18,315
Vill. of Minnesota Lake (Minn.)	Water	G	2,730
Bel Air, Maryland	Sewage	L and G	124,750
Clear Spring, Maryland	Water	L and G	23,000
Vienna, Maryland	Water and Sewage	L and G	27,000
Springfield, Illinois	Water	L and G	1,385,000
Albion School, Ind.	School	G	3,700
Cincinnati, O.	Sewage	G	8,850
Cincinnati, O.	Storm sewer	G	160,000
Cincinnati, O.	Sewage	G	8,730
Cincinnati, O.	Roads	G	27,000
Huntingburg, Ind.	Water	G	84,750
Cleveland Met. Park Section	Parks	L and G	650,000
Salem, Mass.	Streets	G	34,800
Colorado, State of	Highway	L and G	10,000,000
Merrill Water Co., Oregon	Water	L	17,000
Tampa, Florida	Drydock and Shipyards	L	750,000
Colome, S. D.	Water	L and G	25,500
Augusta, Ga.	School	L and G	710,000
Green Bay, Wis.	Sewer	L and G	722,907
Oshkosh, Wis.	Sewer	L and G	968,000
Marlborough, Mass.	Sewer	L and G	25,000
Springfield, Ohio	Sewer	L and G	904,364
Red River Bridge, Ark.	Bridge	G	20,000
Detroit, Mich.	Sewer	G	21,000
Detroit, Mich.	Streets	G	300,000
Omaha, Neb.	Water and Gas	G	290,256
North Kingston, R. I.	Water	L and G	110,000
Total			$29,634,652

Many Cities Busy Organizing for Housing Projects

THE PAST WEEK has witnessed active financial and organizational planning on the part of nearly all of the cities that have received promises of loans from the PWA for low cost housing. In Cleveland, Leyton E. Carter, director of the Cleveland Foundation, was made president of Cleveland Homes, Inc., the limited-dividend company that will spend $14,000,000 on several projects. A campaign is underway to raise the $2,000,000 of equity money required. The Ohio state housing board created the Cleveland Metropolitan Housing District, embracing nearly all of Cuyahoga County, and sent notices to the four appointing powers to name the members. One member each is to be named by the probate court, the common pleas court and the board of county commissioners, while the Mayor of Cleveland names two. This is the first public housing board in the county with authority to build or operate private dwellings.

Slum project in Indianapolis

In Indianapolis a limited-dividend corporation is being planned by the community plan committee of the Chamber of Commerce to administer the $4,460,000 loan granted by PWA to clear up two slum districts and build single family homes, row houses, and three-story apartments in a project designed mainly for occupancy by negroes.

In Raleigh, N. C., citizens are raising some $39,000 to go with the government loan of $168,000 for a project of non-fireproof, three-story buildings to be located on an acre of ground midway between the center of the town and the state university. The development will be planned to be used by state employees and teachers and students of the university.

In New York City, several projects are actively in the planning stage. That of the Hillside Housing Corp. in the Bronx will cover a site of 14½ acres. The PWA loan is $5,184,000. The limited-dividend corporation known as Hallets Cove Garden Homes, Inc., which received a $2,965,000 loan, will build 31 six-story apartments on an 8-acre site in Queens, fronting on the East River and in close proximity to the new Triborough Bridge.

Unique plan in Euclid

In Euclid, Ohio, where a $1,000,000 loan was granted two weeks ago, the Euclid Housing Corp., sponsored by the city government, is receiving applications from citizens owning lots that are not mortgaged. Such owners can secure a loan from the PWA loan with which to build a home. No local money has to be raised, the lots serving as the equity. Construction of groups of 25 or more houses will be let to responsible builders. Euclid is an industrial suburb of Cleveland, and many em-

ployees of nearby manufacturing plants own lots in the city.

In Detroit, the city council approved plans for applying for a federal loan for a 10-block slum-clearance housing project bounded by Rowens, Hastings, Wilkins and St. Antoine Sts. on the east side of the city. The city will purchase the area at 1933 assessed valuation of land and buildings ($845,970) and will clear it ($136,000) and construct the new buildings ($1,970,000). The total cost therefore is placed at $2,951,970.

Chesapeake Bay Region Plans Coordinated Development

In order to put into operation a practical plan for improving sanitary conditions in Chesapeake Bay and its tributaries, the Chesapeake Bay Authority has been organized and Dr. Thomas S. Cullen, prominent surgeon of Baltimore and a member of the staff of Johns Hopkins University, has been appointed chairman. Dr. Cullen has been actively interested in improving sanitary conditions in Chesapeake Bay for the past 20 years. At the present time he is a member of a special advisory committee, which is assisting George L. Radcliffe, regional adviser of the Emergency Public Works Administration for District No. 10, comprising Delaware, Maryland, Virginia, West Virginia, Kentucky, Tennessee, and North Carolina. In addition to coordinating plans for sewage disposal and water supply in the area tributary to Chesapeake Bay, the Chesapeake Bay Authority will give attention to such matters as soil erosion and land use.

The Chesapeake Bay Authority's operations will be closely coordinated with the work of Mr. Radcliffe, the regional adviser. Recently a conference was held in Mr. Radcliffe's office, at which representatives of the Chesapeake Bay Authority met with representatives of several of the states to discuss the relation of proposed public works projects to the development of the region as a whole. A similar conference is to be held this week.

PWA Aid for Sewerage Projects Sought by Hackensack Authority

The Hackensack River Sewerage Authority, recently organized under an act passed by the New Jersey legislature, plans to apply to the Public Works Administration for a combined loan and federal grant of about $25,000,000 to finance construction of the proposed trunk sewer system and sewage treatment plants for the Hackensack River Valley. Thomas J. Wasser, chairman of the Hackensack Valley Sewerage Commission, is chairman of the new Hackensack River Sewerage Authority. In 1931 George W. Fuller and S. Wood McClave, engineers for the commission, prepared the plans which will serve as the basis for the loan application. The river valley has an area of 145 square miles in the state of New Jersey and 62 miles in New York State. The section in New Jersey includes about 50 communities with a population of 450,000. The New York State portion of the valley has a population of only 24,000.

Los Angeles Terminal Site Approved After Controversy

Settlement of the twenty-year controversy over construction of a union railway terminal at Los Angeles, Calif., has been reached with the selection of a site in the Plaza area, adjoining the civic center. Announcement of the agreement of officers of the Southern Pacific, Union Pacific and Santa Fe railroads with city officials on the site for the proposed station was made Sept. 10 by Mayor Frank L. Shaw. The presidents of the three roads, Angus D. McDonald of the Southern Pacific, C. R. Gray of the Union Pacific and S. T. Bledsoe of the Santa Fe, concurred in the Plaza site selection and agreed to accept the city's offer of $1,000,000 toward construction of grade separations, grading and street paving adjacent to the Plaza. Executives of the railroads previously had favored a North Broadway site for the proposed station.

In his announcement of the settlement, Mayor Shaw said the rail executives would submit to the California railroad commission plans for a structure facing Alameda street, and providing for through traffic, rather than a "stub-end" station. It is anticipated that completion of the project will call for the expenditure of approximately $10,000,000.

Highway to Alaska Favored In Road Commission Report

The American members of a commission appointed some time ago to report on the feasibility of constructing a highway from the northwestern part of the United States through Canada to Alaska is reported to have advised President Roosevelt that construction of such a highway line is feasible. The cost is estimated at $14,000,000, of which $2,000,000 would cover the cost of the American section in Alaska and $12,000,000 the section in British Columbia and Yukon Territory. The total distance from Seattle to Fairbanks is 2,256 miles, of which there is a highway of some sort for 1,073 miles. The Alaska section is 274 miles long and 91 miles of that is in service.

Committee Reports Data on Construction Working Conditions

COMPREHENSIVE data on hours and wage rates in the construction industries have been gathered by a committee of the Construction League of the United States with a view to providing information that will assist in framing the labor requirements of the construction code. They were reported by the committee last week to the League's special committee on the code. The report includes facts on hours actually worked in construction and on wages paic the various skilled trades and unskilled labor during the last few years as well as throughout the period 1913-1931.

Hours and wages are shown to depart far from those prescribed for construction under the Public Works Administration (30-hour week, fixed by Recovery Act; wage rates, set by PWA regulation, $1.00 to $1.20 skilled and 40c. to 50c. unskilled). They also differ markedly from the rates that have been suggested for the code. Working hours were found to exceed 44 hr. per week in the majority of construction operations and cities (figures from about 500 cities). Skilled-labor wage rates in 80 per cent of these cities were found to be below the Public Works Administration scale, and in 42 per cent below 75 cents. Non-union scales were found to range from 16 to 36 per cent below union scales.

In an analysis of the conditions governing working conditions in the construction industries the committee says that while the desirability of establishing wage scales as high as economically justified is recognized, scales higher than warranted by current market conditions discourage private construction operations; this effect would check the recovery of construction. It points out that private construction is an essential element of recovery because the public-works program represents only a small part of the normal volume of construction. New construction is optional rather than necessary, the report states, inasmuch as old structures can be made to serve when the cost of new construction is unduly high.

The committee also presents facts to show that the data on which the PWA rates were based are not complete and representative, and concludes that these rates if applied to private work would raise the cost of improvements so much that investment money would not be attracted.

The committee concludes that current non-union scales more nearly represent economic wage scales than do union scales; that wage rates in small towns and rural districts are generally lower than in cities; that reduced-wage agreements for repair and alteration work, coupled with the existing demand for repair and modernization, indicate that minimum code scales can and should be set low enough to encourage this type of activity; and that wage scales should be brought into line with commodity prices and cost of living, which latter factors remained fairly constant from 1921 to 1930, while wage scales increased by nearly 50 per cent.

Contrasting conditions in the construction field with those in the consumer goods industries the committee points out that in the latter it is possible to raise prices fairly quickly to make up for increased labor costs, while in construction the investor and not the contractor or engineer is the actual employer; his willingness to engage in a venture is directly discouraged by rising costs. The committee urges that this difference in fundamental conditions be given consideration as vital to any plan for increasing construction employment, "which can be most effectively done by giving every possible encouragement to increased construction volume."

The special committee which prepared the report is headed by J. W. Taussig, of the Raymond Concrete Pile Co. Thomas S. Holden of the F. W. Dodge Corp. is vice-chairman.

Fuller details of the report will be given in a subsequent issue.

Power Authority in Tennessee Valley Sets Power Rates

THE Tennessee Valley Authority, on Sept. 14 and 15, announced the rates which it will charge to municipalities and other consumers for power, also the rates which it will expect municipalities to charge consumers of such power. The official announcement reads in part as follows: "Any municipality, in the area we plan to serve initially, which owns its distribution system, assuming half-time use, may secure wholesale power from the Authority at an average cost to it of seven mills a kilowatt hour. We propose that our municipal wholesale customers charge the individual residence consumer in the towns and cities thus served a maximum gross rate of 3 cents a kilowatt hour for the first block, and for subsequent blocks 2 cents, 1 cent, and 4 mills. For the typical general consumer this is an average of about 2 cents a kilowatt hour, and for the typical limited user an average of about 2½ cents. For a fully electrified home—which is our objective—the rate would average 7 mills per kilowatt hour."

Wholesale electricity rates for 1,000 hp. and over, delivered at the secondary side of the substation transformer, main line switch and transformers to be provided by the Tennessee Valley Authority, is to be sold at $22.50 per hp. per year.

Suggested rural rates, corresponding to those set for municipal rates, also have been set as follows:

First 50 kwh	3c. per kwh.
Next 150 kwh	2c. per kwh.
Next 200 kwh	1c. per kwh.
Over 400 kwh	4 mills per kwh.

Minimum monthly bills for cooperatives and for municipally constructed distribution lines are outlined on the basis of the number of customers per mile.

With reference to the wholesale rates announced, the Authority states: "While strikingly low, (the rates) cover all costs of furnishing service, and are based on the Board's policy that Muscle Shoals shall be entirely self-supporting and 'bankable'; in other words, a business enterprise. Computations are based on the best available data and weeks of close study analysis, and have been subjected to the criticism of recognized rate experts. The figures show that all costs of service are included, and, in addition, provision has been made for items of costs not actually incurred, such as taxes and interest, but which were nevertheless, included in order to make fair comparison with privately operated utilities, for 'yardstick' purposes."

Municipal Light Plant Expansion Favored by Cleveland Council

The City Council of Cleveland, Ohio, recently went on record as favoring the submission to the voters in November of a $2,000,000 bond issue for expansion of the municipal light and power plant. While the city is close to its bond-issuing limit, a recently-enacted state law permits the sale of bonds outside the bonding limits if the bonds are to be sold to the Public Works Administrator for work to be financed under the Recovery Act.

Clement Chase Killed in Fall from Philadelphia Bridge

Clement E. Chase, a member of the firm of Modjeski, Masters & Chase, was killed in a fall from the Delaware River bridge at Philadelphia on Sept. 18. Mr. Chase was one of the engineers connected with the design of the bridge and at the time was engaged in making an inspection preliminary to the construction of a high-speed line over the bridge from Philadelphia to Camden. He was swept off the bridge by a strong gust of wind and fell to the waterfront street 120 ft. below, striking against the pier in his fall.

Mr. Chase was born in Omaha, Neb., in 1888, and graduated as a civil engineer from Cornell in 1910. Previous to graduation he had served as a concrete and steel inspector, on the McKinley Bridge at St. Louis under Ralph Modjeski and since graduation has been associated continuously with Mr. Modjeski on bridge work.

Columbia Basin Development Gets Federal Power Permit

The Federal Power Commission has granted to the Columbia Basin Commission a preliminary permit for the construction of its proposed dam on the Columbia River near the mouth of the Grand Coulee. While the commission now plans only to build the dam to part of its ultimate height, the permit covers a dam built to the height required for full development of the site. The permit authorizes the commission to go ahead with its preliminary work and to proceed with the preparation of an application for a full license.

Recently the Administrator of Public Works approved an allotment of $1,000,000 for the construction of the Grand Coulee Dam in conjunction with the PWA's previous tentative allotment of $63,000,000 to the Columbia Basin Commission for the power development project. The money now authorized will go to the Bureau of Reclamation for the construction of a construction camp and for a bridge crossing Columbia River at the site. The present allotment is a loan. When the full allotmen is approved, it will include a 30 per cent grant. At present the larger allotment is tied up in legal difficulties which may involve legislative action.

Niagara Frontier Counties Ask PWA Aid for Sanitation Plan

The Niagara Metropolitan Sewerage Commission, representing communities fronting on Lake Erie and the Niagara River, has applied to the National Recovery Administration for a full grant of $26,000,000 to end pollution of the boundary waters along the Niagara frontier. A full grant of the money required for the work instead of a request for a loan of 70 per cent and a grant of 30 per cent is asked on the grounds that early in the present century the federal government entered into a treaty with Great Britain providing for the maintenance of the boundary waters in a sanitary condition. Arthur S. Tuttle, state engineer for the PWA for New York, has advised the committee that he will be unable to pass the application on to Washington if a full grant is required.

Waite Further Explains PWA Loan Procedure At Chicago Meeting

IMPORTANT further explanations of PWA procedure were given by H. M. Waite, Deputy Administrator, at a conference held Dec. 15 at Chicago with the organizations of the five lake states (Region III, Ohio, Indiana, Michigan, Illinois and Wisconsin) under Charles M. Moderwell, regional adviser. Col. Waite stressed the fact that the $3,300,000,000 voted in the Recovery Act for unemployment is surely going to be spent, and that if the states do not take up for non-federal projects what was now left, about one-half, by Jan. 1, 1934, then the remainder would quickly be allotted to federal projects.

Labor conditions

The conference developed that the greatest difficulty arises from the wage schedule, 50c. for common labor and $1.20 for skilled labor. In Wisconsin a graduated scale beginning at 45c. and advancing to $1.10 has been adopted for public and private work, differentiated for five groups of work and seven sizes of city, but adjustment between this scale and the PWA scale, Col. Waite said, will have to be taken care of through the Federal Labor Board of Review. Intermediate rates (for helpers) between 50c. and $1.20 must be fixed by the engineers, but no definite rule is available to decide what to write into specifications and advertisements for bids as to these rates.

The distinction between labor and material costs and other items of cost on which 30 per cent grant will not be allowed was thoroughly discussed, but aside from the ruling that engineering cannot be counted as part of labor cost this twilight zone was left obscure, and the place of contractor's inspection, clerical help and foremen was not cleared up.

Thirty per cent grant

One other point particularly stressed by Col. Waite was the reason for the 30 per cent grant, which he indicated was not intended to be more than enough to balance the extra costs due to the 30-hour week and the higher hourly rate. Looked at as a weekly wage the PWA wage rate is not particularly high, and the short hours permit double-shift working, thus giving employment to twice the number of men. His experience and subsequent reports led him to believe that contractors are getting nearly as good results from the shorter day as from the longer normal day, and that costs are running 20 to 25 per cent over normal, so that the 30 per cent grant fully covers the extra cost.

Force-account work is not looked on with favor. Essentially everything done under PWA must be by contract, said Col. Waite, since it is next to impossible to check day-labor jobs or estimate them beforehand. All old contracts are past history; on old projects with contracts partly completed new contracts must be written.

Highways are likely to be given a lot of 30-70 money, it was stated; the $10,000,000 allotted to Colorado was cited as example.

Handling of loans to cities to build competing utility plants is a problem. In Wisconsin the PWA engineer is requiring

a permit of convenience and necessity from the State Public Utility Commission before considering the application. The applicant will have to conform to state laws in cases of this kind.

Col. Waite said that it was highly desirable to have advertising done and bids in even before contract is made with PWA. As to general or national planning, while this is likely to make an excellent start it probably cannot be done in time to help much on emergency work. The board for the Mississippi Valley will doubtless be able to coordinate future federal expenditures for flood control, river regulation and development to mutual advantages so as to better allocate funds for the next 25 years.

Construction Code Hearings

Hearings on the supplemental codes for the construction industry were resumed on Sept. 18 in Washington, the code for the plastering industry being taken up at that time. Hearings on the limestone code were scheduled for Sept. 19.

The hearings on the basic code for the industry and on some of the supplemental codes, begun on Sept. 6, were concluded on Sept. 11 without holding a public hearing on the code for the plumbing and heating industry as the hearing on that code was postponed at the request of the National Association of Master Plumbers.

The engineers code has been filed with the Administrator but no date for a public hearing has been set. This code has been printed and copies may be obtained from the Superintendent of Documents, Washington, D.C., price 5 cents. Its registry number is 1719-2-18 and its fall title is "Proposed Code of Fair Competition for the Professional Engineers Division of the Construction Industry."

SOCIETY CALENDAR

AMERICAN ASSOCIATION OF ENGINEERS, annual meeting, Chicago, Ill., Oct. 3-6, 1933.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual meeting, Milwaukee, Wis., Oct. 9-11.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, annual meeting, Chicago, Oct. 17-21.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, Indianapolis, Ind., Oct. 9-12.

AMERICAN SOCIETY OF MUNICIPAL ENGINEERS, annual meeting, Milwaukee, Wis., Sept. 25-27.

AMERICAN WELDING SOCIETY, fall meeting, Detroit, Mich., Oct. 2-6.

HIGHWAY RESEARCH BOARD, National Research Council, Washington, D. C., Dec. 7-8.

NATIONAL SAFETY COUNCIL, 22nd Annual Safety Congress, Chicago, Oct. 2-6.

NATIONAL ASSOCIATION OF RAILWAY AND UTILITIES COMMISSIONERS, annual meeting, Cincinnati, Ohio, Oct. 10-12.

THE WASHINGTON STATE ASSOCIATION OF COUNTY COMMISSIONERS AND ENGINEERS held its annual convention at Wenatchee, Wash., Sept. 1-3. Ross D. Alverson, county engineer of Snohomish County, was named president for the engineers for the coming year.

PENNSYLVANIA WATER WORKS OPERATORS ASSOCIATION held its annual meeting at State College, Pa., Sept. 7-9, following the session of the Pennsylvania Sewage Works Association. A total registration of 167 was recorded. Officers elected for the following year are M. E. Fientje, president, H. M. Freeburn and C. L. Fox, vice-presidents, and I. M. Glace, secretary-treasurer.

Milwaukee Not in Contempt over Activated Sludge Patent

On Sept. 6, 1933, ten days after a federal circuit court at Milwaukee had issued an order in the activated sludge patent case recently brought against the city, the Milwaukee Sewerage Commission took an appeal to the U. S. Circuit Court of Appeals for the Seventh District and obtained a stay of the preliminary injunction. Reports current in Milwaukee would indicate that the Jones Island plant was likely to be closed down, but the "order" is due legal form in patent cases and while technically it demands abatement of infringement the appeal to a higher court, if granted, stays action pending the outcome of the higher court's findings.

This complicated case, which has been in the courts for four years, was tried in May, 1929. On Feb. 9, 1933, Judge F. A. Geiger filed a comprehensive opinion sustaining the patents of Activated Sludge, Inc., and holding them infringed. (See *ENR*, Feb. 23, p. 264.) Pursuant to the opinion a decree was entered in the U. S. District Court at Milwaukee providing among other things for a permanent injunction and an accounting of profits and damage accruing by reason of the infringement by the city of Milwaukee. The city appealed the case. The printing of the voluminous record has been completed and the case has been ordered to be heard in the October term of the Circuit Court of Appeals.

Attorneys for the city assigned as error the finding of validity and infringement as against the city, while it is claimed that the responsibility was that of the Sewerage Commission. Activated Sludge, Inc., thereupon filed a bill of complaint against the commission and on July 6 obtained the entry of an order for preliminary injunction, being in effect a duplication of the order against the city. Since as noted above both are enjoined and both have appealed, the matter now rests in the Appellate Court.

Concurrent with the Milwaukee cases is that of the Activated Sludge, Inc., against the Sanitary District of Chicago. The original bill of complaint was filed Sept. 19, 1924, charging infringement in the construction and operation of the several plants serving Chicago. The Milwaukee case it will be seen was started after this Chicago case.

The Chicago case was tried at final hearing before Judge Walter C. Lindley, the trial beginning April 4, 1933, and lasting many weeks. Upon completion of testimony, including depositions taken in England and several places over this country, the case is to be argued this fall.

Much confusion has arisen in the Milwaukee case because in the original case the city was named as defendant, rather than the commission. The original order lay against the city, which the complainant held is the parent of the commission. Eventually a new case was instituted against the latter, and a similar decision and order was given by Judge Geiger against the commission, which as noted above, filed an appeal just as the city had done.

Suit has recently been filed against both city of Indianapolis and sewerage commission there, but it probably will not be pushed pending the outcome of the Chicago and Milwaukee cases.

Mexican Senate Approves Treaty for Straightening Rio Grande

The Mexican Senate, on Sept. 13, ratified the treaty between the United States and Mexico providing for straightening the Rio Grande along the international boundary line from El Paso, Texas, to Box Canyon below Fort Quitman, a distance of 155 miles.

Plans for this river straightening were prepared by the International Boundary Commission set up by the two governments for that purpose. Details of them were given in *Engineering News-Record*, Feb. 16, 1933, p. 226. L. M. Lawson is commissioner for the United States and C. M. Ainsworth consulting engineer. The work is estimated to cost $6,106,500, including the construction of a detention dam.

The United States Senate ratified the agreement on April 25. An application for $5,000,000 for the work has been prepared by the American section of the commission and is now before the PWA.

20-Mile Railroad To Be Built To Mine in New Mexico

Contract for the construction of a 20-mile railroad from a point in New Mexico on the Pecos Valley line of the Atchison, Topeka and Santa Fe Railway to the potash mine being opened by the Potash Company of America has been let by the railroad company to the firm of Sharp & Fellows, of Los Angeles.

Union Station Proposed For Railroads Entering Toledo

The Chamber of Commerce of Toledo, Ohio, has appointed a union station committee with Mr. Charles A. Dana, president of the Spicer Manufacturing Co., as chairman. Mr. Dana has begun negotiations with the railroads to obtain their views on the proposal.

Obituary

CLARENCE F. BELL, engineer of the Board of Water Supply, New York, died at his home there on Sept. 8, age 55 years. Mr. Bell began work with the Board of Water Supply as a draftsman in 1905.

JOHN A. BURKE, vice-president and general manager of the New England Concrete Pipe Corporation, Needham, Mass., died at Boston on Sept. 9 following an automobile accident.

C. T. SYLLIAASEN, senior member of the contracting firm of Sylliaasen & Sando, died at his home in Seattle, Wash., on Sept. 5. Mr. Sylliaasen was born in Norway in 1860 and moved to Seattle in 1901. His son, M. O. Sylliaasen is city engineer of Seattle.

CARL GAYLER, bridge engineer, retired, died in St. Louis on Sept. 1, age 83 years. Mr. Gayler was a native of Stuttgart, Germany, and following his coming to this country served as a technical assistant to Capt. James B. Eads. He served as bridge engineer for St. Louis from 1882 to 1906.

Cleveland's PWA Projects Approved by State Board

The Ohio Advisory Board of the Public Works Administration has approved and will forward to Washington the application of the City of Cleveland for loans and grants for two projects in that city. The projects are a water main installation program costing $3,250,000 and construction of a garbage and refuse incinerator to cost $893,800. A third project for the construction of pre-settling tanks at the easterly sewage disposal plant at a cost of $253,500 also was approved by the board, but the application will not be forwarded to Washington until more engineering data are available. Contracts for the water main program already have been prepared but will not be signed until the federal money is available.

Important Ruling on Contracts Made By Public Works Board

Important information for all who have to do with the calling of bids or with contracts for federal work under the Recovery Act is contained in Bulletin 51, Federal Emergency Administration of Public Works, entitled Information Relating to the Negotiation and Administration of Contracts for Federal Projects under Title II of the National Industrial Recovery Act. The bulletin was issued Sept. 7, copies may be obtained from the PWA or the Superintendent of Documents, Washington, D. C.

Since the bulletin was issued a question has arisen with reference to Part II, Sec. 53 of instructions to bidders on federal projects which stipulates that no bids will be accepted from contractor or subcontractor who has not signed and complied with the code for his industry. Comptroller General McCarl has ruled that the government can force compliance but not signature. This means, in practical effect, that contractors must comply with the code to the extent that it is not displaced by PWA requirements on 30-hour week and minimum rates.

There has been an important change in regulations with reference to labor provisions. Circular No. 1 of the PWA construed the 30-hour week on all projects financed by PWA, whether federal or non-federal, to permit application of a code if any had been adopted for the particular class of labor. That alternative has been stricken out. The 30-hour week and the minimum wage rates agreed to by the trade unions with PWA now apply on all federal and non-federal projects, viz., labor on the project and also on materials and supplies fabricated on site of project. Code provisions apply to fabrication of materials and supplies for project, if the work is done at a location other than site of project.

In these respects the answers to questions on PWA loan procedure published in *ENR*, Sept. 14, p. 324, must be modified.

Georgia Gets Highway Money

Georgia's share of the $400,000,000 for highway construction set up under the National Industrial Recovery Act, was made available to the state early in September, after having been held up since July 1 because of the controversy within the state over the control of the state highway department. Plans for the allocation of the money, as required by the Recovery Act, are being rushed in order to have them approved by the Bureau of Public Roads so that construction work may be started at an early date.

Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Sept., 1932	Weekly Average Four Weeks	Week Sept. 21, 1933
Federal Government	$4,670	$4,245	$13,336
State and municipal	15,464	10,900	10,668
Total public	$20,134	$15,145	$24,004
Total private	2,763	6,002	6,684
Week's total	$22,897	$21,147	$30,688
Cumulative to date:			
1932		$891,796	
1933		635,532	

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

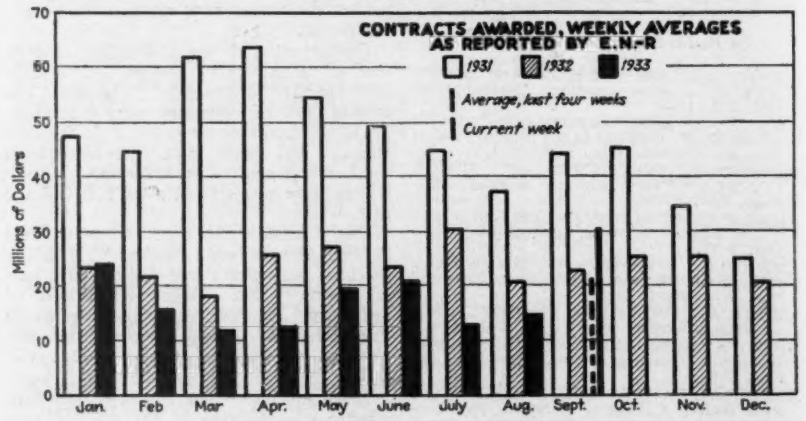
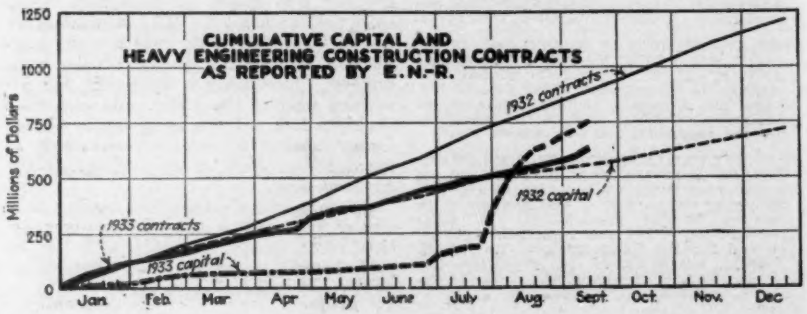
	Ave., Sept., 1932	Week Sept. 21, 1933
State and municipal issues	$3,750	$5,562
Federal PWA loans and grants	.....	30,885
Total	$3,750	$36,447
Cumulative data:		
Total 1932		$528,850
State and municipal	$214,233	.....
PWA loans and grants	539,685	.....
Total 1933		$753,918

Note—These new capital figures include all private bond and stock issues sold for new productive purposes; all state and municipal bonds sold for construction purposes; all R.F.C. advances made on previous commitments on self-liquidating loans; and loans and grants made by the Public Works Authority to states, municipalities and private corporations for new construction work, including the special highway fund, but excluding all federal allotments from the PWA fund for federal projects that would ordinarily be financed by regular federal appropriations.

ENR Cost and Volume Index

E.N.-R. Cost		E.N.-R. Volume	
September, 1933	175.48	August, 1933	73
August, 1933	167.00	July, 1933	65
September, 1932	157.96	August, 1932	111
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1930 (Average)	202.35	1930 (Average)	260
1913 (Average)	100		

Engineering Contracts and Capital

THE LONG-AWAITED federal public works program was started last week with government contract awards totaling $13,336,000, nearly all for revetments and dikes along the Mississippi River in Missouri. Except for the Staten Island Marine hospital, noted last week, this is the first federal award of any consequence since last March. The spurt in federal contracts pushed the total of all heavy engineering construction awards to $30,688,000 for the week, a total surpassed only twice this year, once in May when heavy contracts on the San Francisco-Oakland bridge were let and once in January when the Metropolitan Water District took over $19,000,000 in tunnel work on the Colorado River aqueduct. State and municipal awards last week amounted to $10,668,000, of which $8,602,000 was in highway lettings. Highway awards dropped about 2 millions from the previous week, but are well above the average of the past month. Private contracts amounted to $6,684,000, the highest in three weeks. A large number of industrial building contracts, none over $250,000, accounted for most of the private work. The average of all contracts for the past four weeks is $21,147,000, which approaches the Sept., 1932, weekly average of $22,897,000.

New productive capital reported for the week amounted to $36,447,000, of which $30,885,000 was federal PWA loans and grants and the remainder state and municipal bond issues.